UNIVERSITY OF ALBERTA

IN SITU ROUGHNESS MEASUREMENTS OF WATER TRANSMISSION MAINS

by

(C)

Jorge Luiz Andrade Cavalcante

A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment of the requirements for the degree of Master of Science.

in

ENVIRONMENTAL ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA
SPRING, 1992

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ISBN 0-315-73238-5

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DEGREE: Master of Science

YEAR THIS DEGREE GRANTED: 1992

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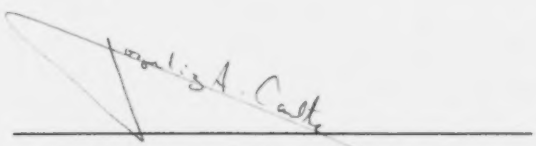

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The undersigned certify that they have read, and recommended to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled *In Situ Roughness Measurements of Water Transmission Mains* submitted by Jorge Luiz Andrade Cavalcante in partial fulfillment of the requirements for the degree of Master of Science in Environmental Engineering.

Gordon R. Finch - Supervisor

February, 19, 1992

To my wife for the support and love received

during the development of my thesis

and to my parents for the privilege of being here.

ABSTRACT

The present study was developed to fill a gap in Hazen-Williams C-factor measurements for large diameter water transmission mains in the City of Edmonton, Alberta. C-factors in small diameter pipes are normally measured by developing head loss tests using hydrants. However, this method does not apply to large diameter pipes. Therefore, procedures to allow C-factor measurement in large diameter water transmission mains were developed. Four parameters were obtained to calculate the pipeline C-factor: head loss, flow rate, pipe diameter, and length. Head loss was obtained by measuring the hydraulic grade line between two consecutive control valve chambers in the tested pipeline. Flow rates were obtained using an insertion sensor style meter. Pipe diameter and length were obtained from "as built" data. This study also defined the necessary procedures to develop the tests and requirements for pipe test section selection. Three lines were studied: one unlined steel pipe with diameter 1050 mm and 16 years old, and two concrete pressure pipes: one with diameter 600 mm and 2 years old and one with 1050 mm and 11 years old. The average C-factor obtained for the steel line was 108, matching the values recommended by technical literature. C-factors for concrete pressure pipes were always above 150, showing that this type of pipes normally operates with smooth flow. C-factors obtained were compared the values used by the City of Edmonton Public Works in their water distribution network simulation model. Low C-factors were used for concrete pipes and high C-factors were assigned to metallic pipes. This misuse was attributed to the need for model calibration where C-factors were adjusted to match pressures observed in the field. This study demonstrated the need for field verification of C-factors when using water distribution system simulation models. The protocol developed and tested in this study can be used to collect some field data in the City of Edmonton.

ACKNOWLEDGEMENT

This work was developed under the financial support of the Water Branch of the City of Edmonton Public works. I here express my sincere gratitude to all departments of the Water Branch of the City of Edmonton Public Works involved in this research that promptly helped in the development of all field tests. Special thanks, I owe to Mr. Ron Heino and Mr. Shaun O'Farell from the Water Network Department of the Water Branch, and to Mr. Clayton Tiedemann from the Maintenance and Operation Department of the Water Branch. The calibration of the equipment had the cooperation of Mr. Sheldon Lovell from the Hydraulics Laboratory, Mr. Gerry Cyre from the Geotechnical Laboratory, and Mr. Roy Gitzel from the Electronics Laboratory in the Civil Engineering Department of the University of Alberta. The most sincere gratitude I owe to those who closely helped me at the most difficult hours: my supervisor, Dr. Gordon R. Finch, and the Chairman of the Civil Engineering Department, Dr. Daniel W. Smith.

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LIST OF NOMENCLATURE

A	-	cross sectional area of flow
A_i	-	annular area corresponding to i th velocity
a,b	-	generic exponents
C	-	Hazen-Williams friction factor
C_e	-	equivalent Hazen-Williams friction factor (used on velocity profile method for determination of C-factors)
C_B	-	Hazen-Williams friction factor for the top half of a pipe (used on velocity profile method for determination of C-factors)
C_T	-	Hazen-Williams friction factor for the bottom half of a pipe (used on velocity profile method for determination of C-factors)
C_o	-	Hazen-Williams friction factor for velocity v_o
C_c	-	Chezy friction factor
C_K	-	generic coefficient related to friction factor
C_m	-	Mills friction factor
C_n	-	Kutter and Manning friction factor
C_S	-	Scobey friction factor for concrete pipes
C_{ST}	-	Scobey friction factor for steel pipes
C_f	-	Darcy friction factor
D	-	pipe diameter
g	-	gravitational acceleration
h	-	total head loss
h_f	-	friction head loss
i	-	number of observations
K_K	-	von Karman constant
l	-	autocorrelation lag
L	-	length of conduit
N_R	-	Reynolds number
P	-	pressure
Q	-	volumetric flow rate
r	-	pipe radius
R	-	hydraulic radius
S	-	hydraulic gradient ($S=h_f/L$)
v	-	average velocity
v_o	-	initial velocity
v_B	-	average velocity in the bottom half of a pipe
v_T	-	average velocity in the top half of a pipe
v_*	-	shear velocity
v_{max}	-	maximum velocity
V	-	Voltage

X	-	generic distance
X_T	-	total length of insertion of the MSR Magmeter probe
y	-	distance from pipe wall
Z	-	elevation
Δ_h	-	difference between height of columns of manometer fluid
Δp	-	differential pressure
α	-	kinetic energy correction factor or Coriolis coefficient
δ_i	-	autocorrelation coefficient
ε	-	equivalent surface roughness
γ	-	specific weight
ν	-	kinematic viscosity
μ	-	dynamic viscosity
ρ	-	fluid mass density
τ_o	-	wall shear stress

1. INTRODUCTION

1.1. GENERAL

Pipeline carrying capacity decreases with time as a result of an increasing pipe roughness. Many coefficients have been proposed to represent numerically the carrying capacity of a pipeline. However, the Hazen-Williams C-factor is one of the most common friction factor coefficients used in pipeline analysis and calculations due to its simplicity.

The City of Edmonton Public Works has performed flow tests to measure C-factors in the Edmonton's water network. The obtained values were representative for small diameter pipes, but the method was not feasible for larger mains. There is a lack of data for water transmission mains with diameters larger than 500 mm. The C-factors used for system modelling and design of the transmission mains are often not representative of the actual field situation.

The goal of the study reported here is to fill the gap concerning large diameter pipelines. This will give to the City of Edmonton a basis for comparison of the actual roughness in these pipelines with the assumed values, as well as tools for the development of future tests.

1.2. OBJECTIVES

The objectives of this research program were:

1. identification of possible methods to measure C-factors in large diameter pipelines;
2. development of field tests to obtain actual C-factors in pipelines in the Edmonton's water distribution network;
3. analysis and critique of the results by comparing the observed values with those reported in the technical literature;
4. make recommendations on the use of the obtained values in modelling and design, and;
5. preparation of a field protocol for future tests in pipelines already tested or in new pipelines.

1.3. SCOPE

Field tests for C-factor measurement were a two-phase task. The initial phase corresponded to the background investigation of what was available by using the literature review and local experience. First, it was important to select transmission mains to test. Second, it was necessary to identify available equipment for the

development of the field tests. Also, it was necessary to define the C-factor measurement method to use.

In a second phase, the field tests themselves were developed. This phase was divided in equipment calibration in the laboratory and the C-factor measurements in the field. Concluding the field tests, the results were analyzed and compared with other reported results. Finally, the use of the results in network modelling and design was discussed.

2. LITERATURE REVIEW

2.1. HYDRAULICS OF PIPELINES

2.1.1. Energy Conservation and the Bernoulli Equation

A liquid flowing in a pipeline has three types of energy: kinetic, potential, and internal (Walski, 1984). Jeppson (1976) described these three types of energy as: kinetic energy resulting from the motion of the fluid, potential energy resulting from the elevation of the fluid, and internal energy resulting from the pressure of the fluid inside the pipe. The summation of these three forms of energy is called total energy (Walski, 1984).

Considering a streamline in a pipeline, the liquid flows from a point with a higher total energy to a point with a smaller total energy (Walski, 1984). The difference between the total energy in these two points is transformed in other types of energy (Jeppson, 1976). This energy is non-recoverable from the hydraulic point of view, and is considered as a "loss" (Jeppson, 1976; Walski, 1984). The law of conservation of energy represents the three different types of energy existing in a streamline and the energy transformed or "lost" (Brater and King, 1976). This law is expressed by the Bernoulli equation, as follows:

$$\underset{(A)}{\frac{v_1^2}{2g}} + \underset{(B)}{\frac{P_1}{\gamma}} + \underset{(C)}{Z_1} = \underset{(A)}{\frac{v_2^2}{2g}} + \underset{(B)}{\frac{P_2}{\gamma}} + \underset{(C)}{Z_2} + h \qquad (2\text{-}1)$$

where:

- v = average velocity (m/s)
- g = gravitational acceleration (m/s^2)
- P = pressure (Pa)
- γ = specific weight of the fluid (N/m^3)
- Z = elevation (m)
- h = total head loss (m)

This equation applies only to streamlines in a pipeline, not to the whole pipe flowtube (Walski, 1984). To overcome this problem, Walski (1984) and Lencastre (1987) described the use of the correction factor (α), proposed by Coriolis (1836) (Chow, 1973). This correction factor is applied to the kinetic energy term, and is important when high precision is desired in calculations using the Bernoulli equation (Brater and King, 1976). However, it may be ignored in water distribution systems without introducing important error (Brater and King, 1976; Walski, 1984).

Each term in equation 2-1 can be expressed as a head of water. The summation of the velocity, pressure, and elevation heads (A, B, and C, respectively in equation 2-1) defines the energy head. The summation of pressure and elevation heads (B and C, respectively in equation 2-1) defines the piezometric head (Walski, 1984). These heads define the energy and hydraulic grade lines, respectively (Walski, 1984; Lencastre,

1987). Figure 2-1 presents a schematic of the Bernoulli equation for a pipe with constant diameter conducting a steady flow between two points.

2.1.2. Conservation of Mass and the Continuity Equation

Walski (1984) described the use of the continuity equation in the prediction of velocity distribution. This equation in its differential form allows the calculation of the average velocity by integrating the velocity distribution curve over the cross section area of the pipe. However, the velocity distribution curve in a continuous form is seldom known. In a practical situation, the velocity is known at a few points and the continuity equation is used in its discrete form:

$$v = \frac{\sum_{i=1}^{n} v_i \, A_i}{A} \qquad (2\text{-}2)$$

where:

v = average velocity (m/s)
v_i = velocity at i th observation (m/s)
A = cross sectional area of flow (m^2)
A_i = annular area corresponding to i th observation (m^2)

Walski (1984) described the application of this formula in the calculation of the average velocity in a pipe by measuring the velocity profile. In this case, the total cross-section area of the pipe is divided in a series of smaller annular areas, being the fluid velocity for each of those annular areas measured. The area division may be done in equal area rings, or rings equally spaced. However, the second case requires the measurement of the velocity in more points. Figure 2-2 illustrates the division of a pipe section in a ten point traverse with equally spaced rings for the measurement of the velocity profile.

2.1.3. Rational Head Loss Equations

In the last two hundred years, many different head loss/flow formulas have been accumulated (Hinds, 1946). Most of those formulas have an empirical character, but attempts have been made to obtain a universal and rational formula (Hinds, 1946; Swanson and Reed, 1963).

The rational formulas defined the theory of roughness (Lamont, 1981). Table 2-1 summarizes the evolution of the theory of roughness and friction head loss from Chezy (1775) to Moody (1944).

The basis of all head loss/flow formulas is the expression 2-3 on Table 2-1, presented by Chezy in 1775 (Swanson and Reed, 1963). This expression was originally deduced

Figure 2-1. Schematic of the Bernoulli Equation

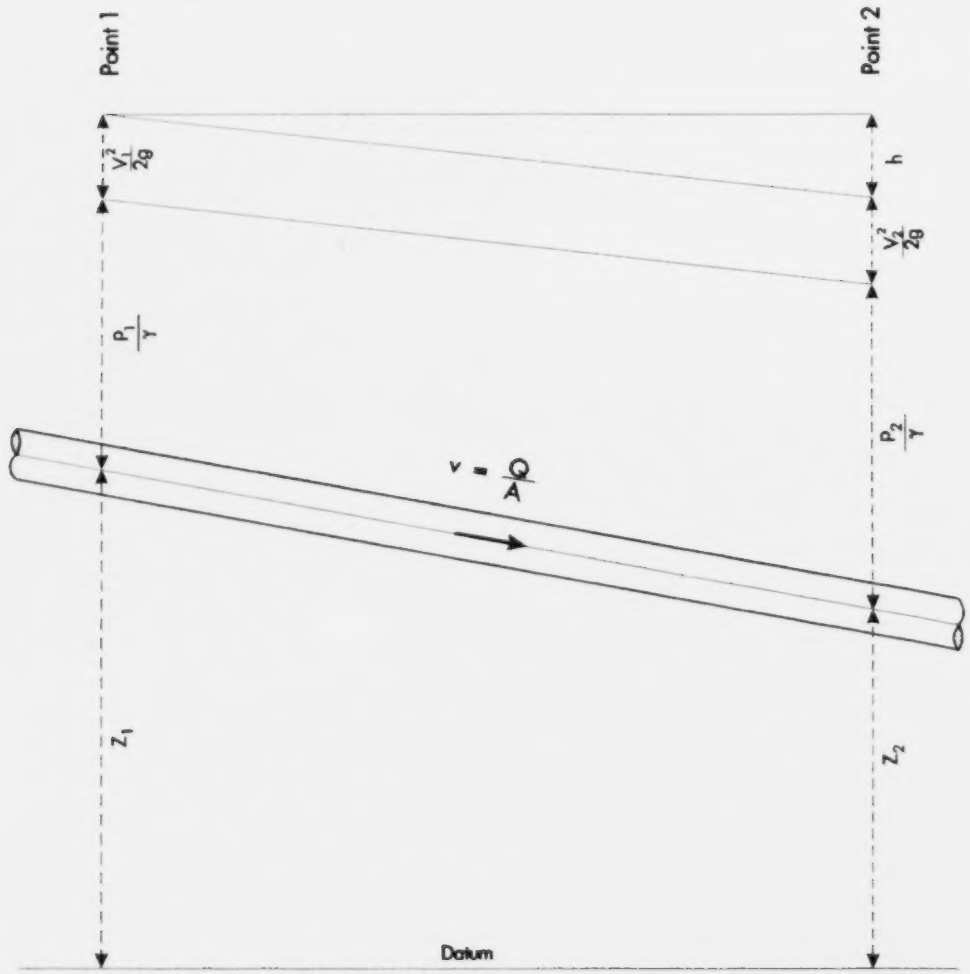


Figure 2-2. Division of Annular Areas for a 10 Point Pipe Traverse

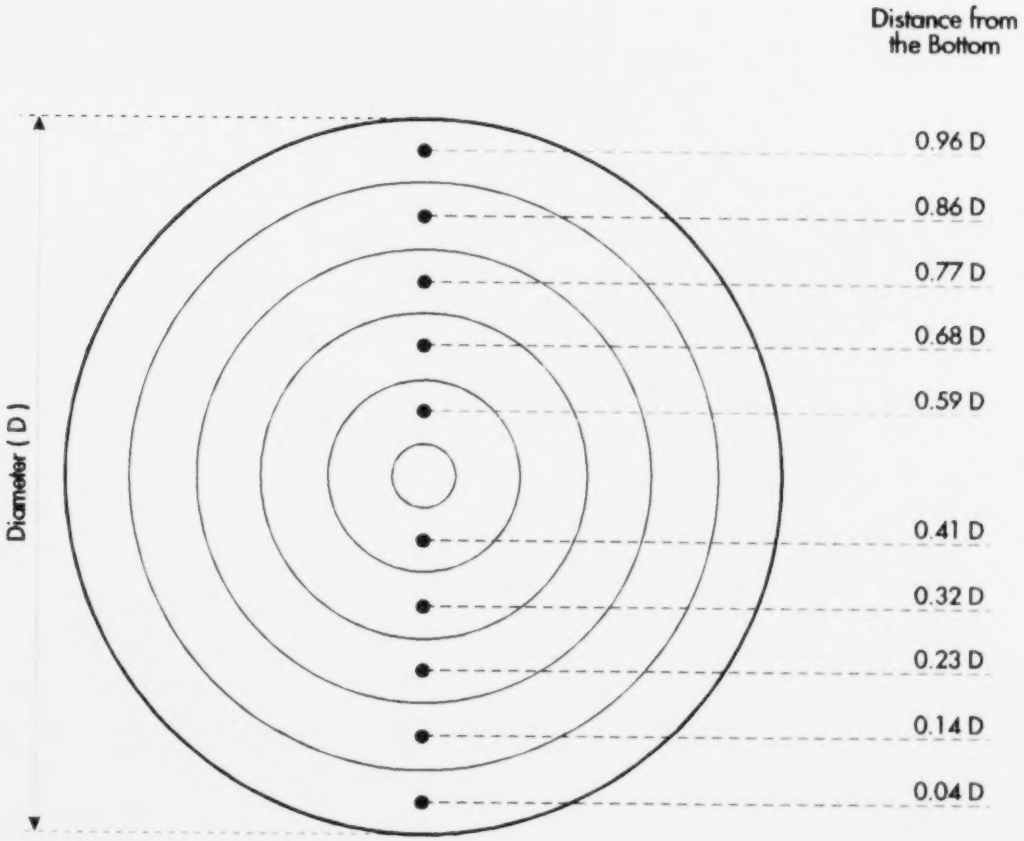


Table 2-1. Evolution of the Theory of Roughness

Year	Name	Equation	Flow Conditions
1775	Chezy [11, 14]	$v = 0.552\ C_c \sqrt{R\ S}$ (2-3)	Flow of water in open channels
1842	Poiseuille [1, 10, 13]	$h_f = \frac{32\ \mu\ v\ L}{g\ D^2}$ (2-4) $\quad C_f = \frac{64}{N_R}$ (2-5)	Laminar flow in pipe
1855 1857	Darcy-Weisbach [2,3,12,13]	$h_f = C_f\ \frac{L}{2g}\ \frac{v^2}{D}$ (2-6)	Pipe flow
1883	Reynolds [4, 13]	$N_R = \frac{v\ D}{\nu}$ (2-7)	Criteria for flow classification
1930 1933	Karman-Prandtl [6,7,9]	$\frac{1}{\sqrt{C_f}} = 2 \log (N_R \sqrt{C_f}) - 0.8$ (2-8)	Turbulent flow in a smooth pipe
1930 1933	Karman-Prandtl [6,7,9]	$\frac{1}{\sqrt{C_f}} = 1.14 - 2 \log \left[\frac{\varepsilon}{D}\right]$ (2-9)	Turbulent flow in a rough pipe
1932 1933	Nikuradse [5, 9, 13]	Systematic tests with controlled roughness	Pipes artificially roughened with sand grains
1938	Colebrook-White [8,9,13]	$\frac{1}{\sqrt{C_f}} = 1.14 - 2 \log \left[\frac{\varepsilon}{D} + \frac{9.35}{N_R \sqrt{C_f}}\right]$ (2-10)	Turbulent flow in smooth, rough and transition conditions
1944	Moody [9]	Plot of $C_f = f\ (N_R, \frac{\varepsilon}{D})$	Moody diagram for commercial pipes for laminar and all turbulent flows

Symbols and Units

- v = velocity (m/s)
- h_f = head loss (m)
- L = length (m)
- S = hydraulic gradient (h_f /L)
- R = hydraulic radius (m)
- D = diameter (m)
- N_R= Reynolds number
- C_c = Chezy friction factor
- C_f = Darcy friction factor
- ε = equivalent surface roughness (m)
- ν = kinematic viscosity (m^2/s)
- μ = dynamic viscosity ($N\ s/m^2$)

Based on:

(1) Poiseuille (1842)
(2) Weisbach (1855)
(3) Darcy (1857)
(4) Reynolds (1883)
(5) Nikuradse (1932; 1933)
(6) Karman (1930)
(7) Prandtl (1933)
(8) Colebrook (1938)
(9) Moody (1944)
(10) Hinds (1946)
(11) Swanson (1963)
(12) ASCE (1965)
(13) Brater and King (1976)
(14) Lencastre (1987)

for flow in open channels (Swanson and Reed, 1963) and was latter generalized for incompressible rough turbulent flow in pipes (Lencastre, 1987).

Poiseuille (1842) developed an equation for the calculation of head loss in laminar flows, as cited by Hinds (1946). This equation, shown on expression 2-4 in Table 2-1 verifies that for laminar flows head loss is independent of the pipe roughness (Brater and King, 1976).

In the middle of the nineteenth century, Darcy (1857) and Weisbach (1855) rewrote the Chezy equation, allowing its use with circular pipes flowing full (Swanson and Reed, 1963; Brater and King, 1976). The Darcy friction factor (C_f) was then introduced, and the Chezy friction factor (C_c) was substituted by the radical $\sqrt{8g/C_f}$, making the Chezy equation dimensionally correct (Hinds, 1946; Swanson and Reed, 1963). The resulting equation is represented by equation 2-6 in Table 2-1.

Brater and King (1976) summarized the studies developed by Reynolds (1883). In those studies, Reynolds (1883) developed a criterion that distinguishes laminar flow from turbulent flow. The expression used for flow classification in circular pipes is represented by equation 2-7 in Table 2-1, and has been called Reynolds number (N_R). According to Brater and King (1976), the flow classification based on the Reynolds number is the following:

- o Laminar flow $N_R \leqq 2{,}000$
- o Transition Flow $2{,}000 < N_R < 4{,}000$
- o Turbulent flow $N_R \geqq 4{,}000$

The calculation of head losses using the Darcy-Weisbach equation depends on the knowledge of the diameter (D) and length (L) of the pipe, average velocity of the fluid (v), gravitational acceleration (g), and Darcy friction factor (C_f). Walski (1984) noted that the first four parameters are easily determined in almost all situations. However, C_f is not directly measured, being different even for a given pipe. Moody (1944) stated that C_f is exclusively a function of two parameters: the Reynolds number (N_R) and the relative roughness of the pipe ε/D (where ε is the equivalent surface roughness and D is the pipe diameter).

Hinds (1946) summarized the methods for determination of the friction factor C_f for different flow classifications. The procedures for the determination of C_f were founded on the studies of Poiseuille (1842), Karman (1930), Prandtl (1933), Nikuradse (1932; 1933), and Colebrook (1938).

Hinds (1946) and Stephenson (1976) considered the calculation of C_f for the zone of laminar flow and the zone between laminar and turbulent flow as unimportant for water

distribution systems. The latter flow zone was classified as uncertain because flow in this region is considered neither laminar nor turbulent (Hinds, 1946). Indeed, these types of flow are a result of very low velocities or pipe diameters, not found in water distribution systems. This is true in most situations, but the advent of PVC may become those regions important in some circumstances.

Hinds (1946) considered the calculation of C_f in the turbulent flow zone as the most important because it represents the actual situation of most water distribution systems. Brater and King (1976) described that turbulent flows are classified in three categories: flows in smooth pipes, rough pipes, and transition between the first two categories.

Karman (1930) and Prandtl (1933), based on tests reported by Nikuradse (1932; 1933), developed equations for the calculation of C_f for flows in smooth and rough pipes (Moody, 1944). These equations are represented by expressions 2-8 and 2-9 in Table 2-1, respectively. C_f for flow in smooth pipes depends only on the Reynolds number (N_R) while C_f for flow in rough pipes depends only on the relative roughness (ε/D) (Stephenson, 1976).

Moody (1944) described the attempts to develop an equation for the calculation of C_f for flows in the transition zone, using the values of Nikuradse (1932; 1933). However, the results obtained were inadequate, not matching the results of the laboratory tests with those encountered in practice. This problem was solved by the equation of Colebrook (1938), represented by expression 2-10 in Table 2-1. This equation covers flows both in the smooth and rough pipe regions as well as the transition zone (Moody, 1944; Stephenson, 1976). C_f for flow in this zone depends on both the Reynolds number (N_R) and the relative roughness (ε/D) (Walski, 1984).

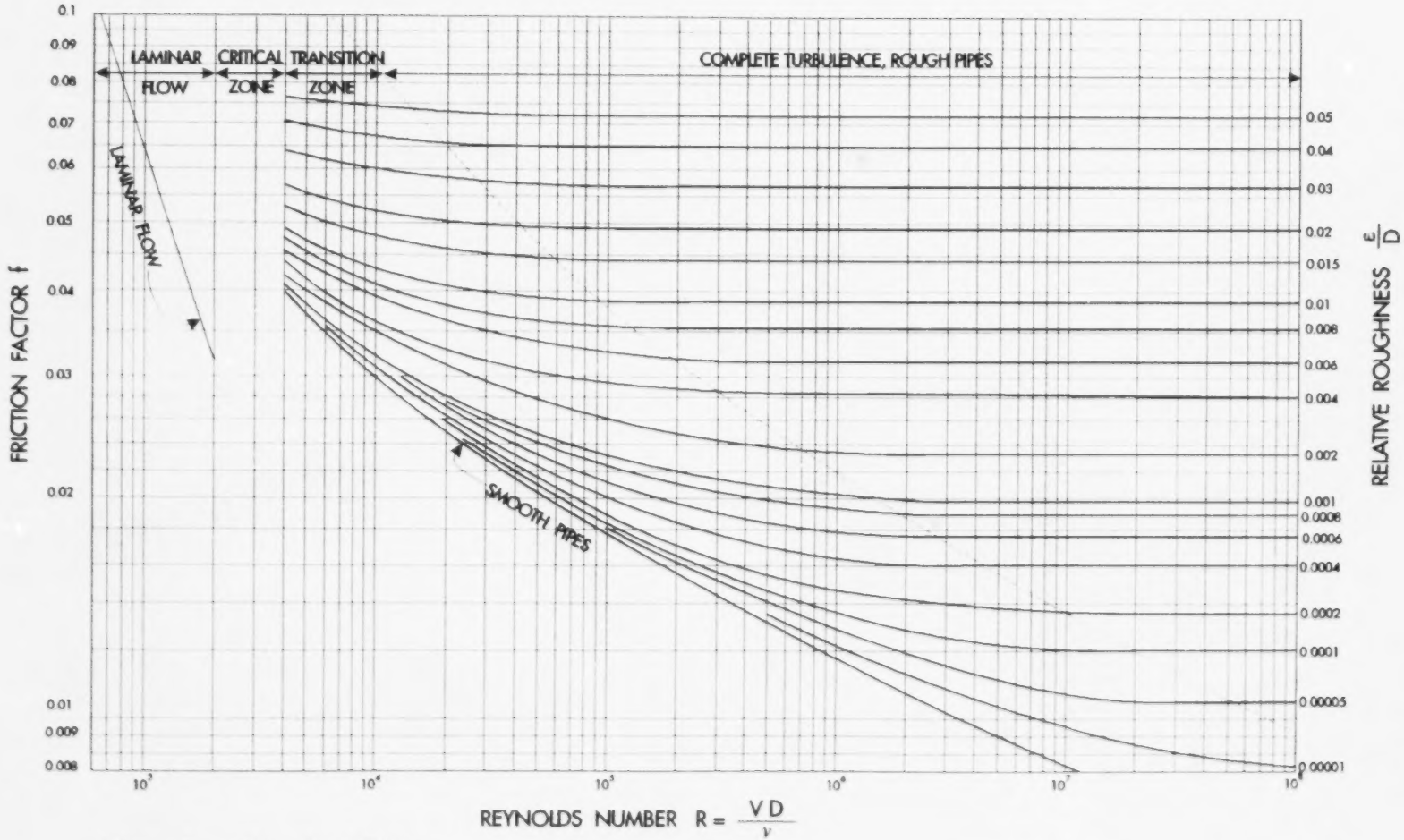
The calculation of the friction factor C_f is complicated with one equation for each type of flow (Hinds, 1946). To overcome this problem, Moody (1944) presented a diagram (Figure 2-3), which received his name. Moody's diagram covers all flow classifications as a function of the Reynolds number (N_R) and the relative roughness (ε/D). Table 2-2 lists values of the equivalent surface roughness (ε), as summarized by Lamont (1981).

2.1.4. Empirical Head Loss Equations

Table 2-3 presents a chronological summary of the most important empirical equations used in the calculation of head loss in water distribution systems. Tables 2-4 to 2-10 illustrate the values used as friction factor for each of these equations.

Stephenson (1976) observed that the first head loss/flow relationships were based exclusively on field observations. Furthermore, those formulas were not dimensionally correct (ASCE, 1965) and depended on the units used (Stephenson, 1976). The general

Figure 2-3. Moody Diagram



(Based on Moody, 1944)

Table 2-2. Values of Equivalent Surface Roughness

Type of Material of Conduit	Equivalent Surface Roughness	
	Mean Value (mm)	Design Value (mm)
Uncoated cast-iron	0.226	0.250
Coated cast-iron	0.102	0.125
Coated spun-iron	0.056	0.050
Galvanized-iron	0.102	0.125
Wrought iron	0.050	0.050
Uncoated steel	0.028	0.040
Coated steel	0.056	0.050
Uncoated asbestos-cement	0.028	0.040
Coated asbestos-cement	virtually smooth	smooth pipe
Smooth pipe	smooth pipe	smooth pipe
PVC pipe (with waviness)	0.030	0.040
Prestressed concrete pipes	0.030	0.040
Spun concrete pipes	0.200	0.250
Pipes relined with cement mortar	0.510	0.500
Concrete miscellaneous		
Scobey		
Class 1 - $C_S = 0.27$ (1)	5.100	5.000
Class 2 - $C_S = 0.31$ (2)	1.270	1.250
Class 3 - $C_S = 0.345$ (3)	0.410	0.500
Class 4 - $C_S = 0.37$ (4)	0.180	0.250
Best - $C_S = 0.40$ (5)	0.102	0.125
Concrete tunnel linings		
Best recorded	0.025	
Practical minimum	0.062	0.062
Mean	0.310	0.310

(based on Lamont, 1981)

(1) Old concrete pipes, mortar not wiped from joints.

(2) Modern dry mix pipes, monolithic pipes, or tunnel linings made over rough forms

(3) Small wet mix pipes in short lengths, dry mix pipes in long lengths, average monolithic pipes made on steel forms, pressure-made pipes with interior coat of neat cement by mechanical trowel.

(4) Monolithic pipes with joint scars and all irregularities removed.

(5) Premium pipes for municipal pipelines conveying reasonably clear water; smooth concrete pipes produced by vibration in rigid oiled steel forms or by centrifugal spinning under definite specifications.

Table 2-3. Chronological Summary of Empirical Head Loss Equations

Year	Name	Equation	Comments
1869	Kutter-Ganguillet[1,2,10,14]	$v = \dfrac{41.65 + \dfrac{1.811}{C_n} + \dfrac{0.00281}{S}}{1 + \left[41.65 + \dfrac{0.00281}{S}\right]\dfrac{0.552\ C_n}{\sqrt{R}}} \; 0.552\sqrt{R\ S}$ (2-12)	Flow of water in open channels, latter adapted to pipes[4]. Coefficient C_n shown on Table 2-4.
1889	Manning[3,14,15,16]	$v = \dfrac{1}{C_n} R^{2/3} S^{1/2}$ (2-13)	Turbulent flow of water in fully rough channels and pipes[17]. Coefficient C_n shown on Table 2-4.
1905	Hazen-Williams[8,12,15]	$v = 0.85\ C\ R^{0.63}\ S^{0.54}$ (2-14)	Turbulent flow of water in smooth pipes[17]. Coefficient C for v=0.9 m/s shown on Table 2-5.
1915	Scobey Wood Stave Pipes[4,10,13]	$v = 49\ D^{0.65}\ S^{0.555}$ (2-15)	Turbulent flow of water in smooth pipes.
1920	Scobey Concrete Pipes[5,10,12,13]	$v = 227\ C_S\ R^{0.625}\ S^{0.5}$ (2-16)	Turbulent flow of water in rough pipes[17]. Coefficient C_S shown on Tables 2-6 and 2-7.
1920	Scobey Steel Pipes[7,10,11,13,17,18]	$v = 23\ \dfrac{D^{0.58}\ S^{0.526}}{C_{ST} 0.526}$ (2-17)	Turbulent flow of water in smooth[17] pipes. Coefficient C_{ST} shown on Table 2-8.
1923[6] 1941[9]	Mills/Capen[6,9,12,13]	$v = 75\ C_m\ R^{0.625}\ S^{0.5}$ (2-18)	Turbulent flow of water in rough pipes. Coefficient C_m shown on Tables 2-9 & 2-10.
1950	Scobey Commercially Smooth Pipes[13]	$v = 47\ D^{0.632}\ S^{0.526}$ (2-19)	Turbulent flow of water in smooth pipes.

Symbols and Units

v = velocity (m/s)
S = hydraulic gradient (h_f /L)
h_f = friction head loss (m)
L = length (m)
R = hydraulic radius (m)
D = diameter (m)

C_n = Kutter & Manning friction factor
C = Hazen-Williams friction factor
C_S = Scobey friction factor for concrete
C_{ST} = Scobey friction factor for steel
C_m = Mills friction factor

Based on:
(1) Kutter and Ganguillet (1869)
(2) Hering and Trautwine (1888)
(3) Manning (1891)
(4) Scobey (1916)
(5) Scobey (1920)
(6) Mills (1924)
(7) Scobey (1930)
(8) Williams and Hazen (1933)
(9) Capen (1941)
(10) Hinds (1946)
(11) Davis (1952)
(12) Swanson (1963)
(13) Scobey (1966)
(14) Chow (1973)
(15) ASCE (1975)
(16) Brater and King (1976)
(17) Walski (1984)
(18) AWWA (1987a)

Table 2-4. Values of the Manning and Kutter Friction Factor C_n

Type of Material of Conduit	Friction Factor C_n			
	Variation		Design	
	Minimum	Maximum	Minimum	Maximum
Clean uncoated cast-iron pipe	0.011	0.015	0.013	0.015
Clean coated cast-iron pipe	0.010	0.014	0.012	0.014
Dirty or tuberculated cast-iron pipe	0.015	0.035		
Riveted steel pipe	0.013	0.017	0.015	0.017
Lock-bar and welded pipe	0.010	0.013	0.012	0.013
Galvanized-iron pipe	0.012	0.017	0.015	0.017
Brass and glass pipe	0.009	0.013		
Wood-stave pipe	0.010	0.014		
Wood-stave pipe, small diameter			0.011	0.012
Wood-stave pipe, large diameter			0.012	0.013
Concrete pipe	0.010	0.017		
Concrete pipe, with rough joints			0.016	0.017
Concrete pipe, "dry mix," rough forms			0.015	0.016
Concrete pipe, "wet mix," steel forms			0.012	0.014
Concrete pipe, very smooth			0.011	0.012
Vitrified sewer pipe	0.010	0.017	0.013	0.015
Common clay drainage tile	0.011	0.017	0.012	0.014
Corrugated metal (2 2/3 x 1/2)	0.023	0.026		
Corrugated metal (3 x 1 and 6 x 1)	0.026	0.029		
Corrugated metal (6 x 2 structural plate)	0.030	0.033		
Rock, unlined	0.038	0.041		
Enamed steel	0.009	0.010		

(based on Brater and King, 1976)

Table 2-5. Values of Hazen-Williams Friction Factor C for a Velocity of 0.9 m/s

Type of Pipe	Diameters					
	25 mm	75 mm	150 mm	300 mm	600 mm	1200 mm
Uncoated cast iron:smooth & new		121	125	130	132	134
Coated cast iron: smooth & new		129	133	138	140	141
30 years old						
slight attack		100	106	112	117	120
moderate attack		83	90	97	102	107
appreciable attack		59	70	78	83	89
severe attack		41	50	58	66	73
60 years old						
slight attack		90	97	102	107	112
moderate attack		69	79	85	92	96
appreciable attack		49	58	66	72	78
severe attack		30	39	48	56	62
100 years old						
slight attack		81	89	95	100	104
moderate attack		61	70	78	83	89
appreciable attack		40	49	57	64	71
severe attack		21	30	39	46	54
Miscellaneous						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron: smooth& new		137	142	145	148	148
Old-take as coated cast iron of same age						
Galvanized iron: smooth & new	120	129	133			
Wrought iron: smooth & new	129	137	142			
Coated steel: smooth & new	129	137	142	145	148	148
Uncoated steel: smooth & new	134	142	145	147	150	150
Coated asbestos-cement: clean		147	149	150	152	
Uncoated asbestos-cement: clean		142	145	147	150	
Smooth pipe (lead, brass, poly-thene, PVC smooth): clean	140	147	149	150	152	153
PVC wavy: clean	134	142	145	147	150	150
Concrete Scobey						
Class 1- $C_S = 0.27$ *		69	79	84	90	95
Class 2- $C_S = 0.31$ *		95	102	106	110	113
Class 3- $C_S = 0.345$ *		109	116	121	125	127
Class 4- $C_S = 0.37$ *		121	125	130	132	134
Best - $C_S = 0.40$ *		129	133	138	140	141
Tate relined pipes: clean		109	116	121	125	127
Prestressed concrete pipes: clean				147	150	150

(based on Lamont, 1981)

* Definition of concrete Scobey's classes on observations of Table 2-2

Table 2-6. Values of Scobey Friction Factor C_S for Concrete Pipes (pipes manufactured according to modern processes)

Type of Concrete Pipe	Scobey C_S		
	Minimum	Average	Maximum
Cast	0.375	0.406	0.444
Centrifugated	0.395	0.409	0.432

(based on Swanson and Reed, 1963)

Table 2-7. Values of Scobey Friction Factor C_S for Concrete Pipes (pipes manufactured according to old processes)

Class	Condition	Scobey C_S
1	Old California cement pipes; generous supply of mortar in joints, mortar squeeze not removed Also: class 2 pipes conveying sewage	0.267
2	Dry-mix precast in short units, washed inside with cement mortar, moderate care; monolithic pipe over rough wood forms; cement gun finish, not troweled	0.310
3	Wet-mix precast in short units; dry-mix precast in long units monolithic pipe on steel forms; small cement-lined iron pipe; concrete pipe made under pressure and mechanically troweled with neat cement. Also: class 4 pipes conveying sewage or detritus-laden water	0.345
4	Glazed-interior pipes; large cement-lined iron pipes; monolithic pipes with joint scars or surface irregularities removed; highest quality precast pipe, made against oiled steel form, with joints as smooth as remainder, untouched with brush or "wash" process	0.370

(based on Davis, 1952)

Table 2-8. Values of the Scobey Friction Factor C_{ST} for Steel Pipes

Class	Condition	Type	Scobey C_{ST}
1	Full-riveted pipe, having both longitudinal and girth seams held by one or more lines of rivets with projecting heads from capacity standpoint; pipe with countersunk rivetheads on interior belongs to class 3	a. New sheet metal up to 3/16" thick	0.38
		b. New plate metal 3/16" to 7/16" thick, with either taper or cylinder joints	0.44
		c. New plate metal 1/2" up, with either taper or cylinder joints, and for plate 1/4" to 7/16" thick, when butt-jointed	0.48
		d. New butt-strap pipe of plate, 1/2" up	0.52
2	Girth-riveted pipe, having no retarding rivetheads in the continuous-seamed longitudinal joints, but having the same girth seams as full-riveted pipe	New sheet- and plate-metal pipe, such as lock-bar and hammer-weld pipe with lap or flange-riveted field (girth) joints; electric weld, hammer weld and drawn pipe with riveted bump joints; and all other types with surface continuous except for girth belt of rivetheads between field units	0.34
3	Continuous interior pipe, having interior surface unmarred by plate offsets or by projecting rivetheads in either longitudinal or girth seams. Not necessarily described as smooth.	New sheet- and plate-metal pipe such as pipe with full-welded crimped slip joint, lock bar with welded flange or leaded sleeve conections, bell and spigot, bolted coupling pipes all belong to this class	0.32

(based on Davis, 1952)

Table 2-9. Values of Mills Friction Factor C_m

Type of Pipe	Value of C_m
Smooth pipe of fairly straight alignment	2.00
Smooth pipe, sinuous alignment	1.80
Fairly smooth pipe, average alignment	1.60
Fairly smooth pipe, sinuous alignment	1.40
Tuberculated pipe or rough pipe	1.20
Pipe greatly deteriorated in capacity and probably requiring relining	1.00

(based on Capen, 1941)

Table 2-10. Comparison of Mills Friction Factor C_m with Other Friction Factors[1],[2]

Pipe Diameter (mm)	For C_m = 1.0		For C_m = 1.5		For C_m = 2.0	
	C	C_m	C	C_m	C	C_m
600	70	0.0180	109	0.0128	148	0.0101
1200	72	0.0192	111	0.0133	152	0.0103
1800	73	0.0198	114	0.0135	155	0.0104
2400	74	0.0202	115	0.0137	157	0.0104
3000	75	0.0204	117	0.0137	159	0.0103
3600	76	0.0205	118	0.0137	160	0.0103

(based on Capen, 1941)

(1) Velocity assumed at 1.22 m/s

(2) C is Hazen-Williams friction factor and C_n is Manning and Kutter friction factor

form was (Hinds, 1946; Stephenson, 1976):

$$v = C_K \ R^a \ S^b \qquad (2\text{-}11)$$

where:

- v = average velocity
- C_K = generic coefficient related to friction factor
- S = hydraulic gradient
- a, b = generic exponents

The friction coefficient C_K and the exponents a and b depend on a series of factors considered by each author in the development of the equation. Therefore, they are normally different for each equation (Swanson, 1963). The controlling element of coefficient C_K is the pipe surface roughness (Hinds, 1946) and generally varies with pipe diameter, conditions of the pipe walls, type of lining, and age of the pipe (Lamont, 1981). The exponents a and b depend on the shape of the curve obtained plotting the experimental points (Hinds, 1946). Walski (1984) reported a criterion to determine empirical equations for turbulent rough pipe flow. In these equations, head loss varies according to the square of the velocity. Therefore, the exponent b of the hydraulic gradient is equal to 1/2. Kutter-Ganguillet, Manning, and Mills/Capen equations, and Scobey equation for concrete pipes confirm this criterion.

The wide use of the Hazen-Williams C-factor in the Northern hemisphere is described by Walski (1984) to be because:

> "Most water system engineers have a very good feel for the meaning of the Hazen-Williams C-factor, while pipe roughness remains a mystery to many practicing engineers."

The adequacy of the Hazen-Williams equation for the calculation of head losses in water distribution systems was demonstrated by Swanson and Reed (1963). Data from different types of concrete pressure pipes were analyzed using the Hazen-Williams equation, Morris equation (Morris, 1955; 1961), Darcy-Weisbach equation, and Scobey equation for concrete pipes. Initially, the average friction coefficients were determined for each pipeline. Using the averaged coefficients, head losses were calculated for each individual test and compared to the observed head losses. Finally, based on the percentage of mean deviation of the head losses, the most applicable equation was defined for each pipeline. Table 2-11 presents a summary of the head loss mean deviation for each pipeline. The Hazen-Williams equation had the smallest mean deviation for the concrete pressure pipes tested.

One difference between the Darcy-Weisbach equation and the Hazen-Williams equation is that the latter does not consider the effect of temperature variation in the fluid. Hinds (1946) described the effect of temperature as irrelevant in water distribution systems. According to Hinds (1946), for a water temperature of 15°C, a variation of ±11°C

Table 2-11. Swanson and Reed Comparison of Deviation of Computed Head Losses from Observed Head Losses

Name and Description of Pipe	No. of Obs.	Inside Diam. (in)	Range of Velocity Min-Max (m/s)	Mean Deviation - per cent			
				Hazen Williams	Morris	Darcy Weisbach	Scobey
Dijon, France, 1895	9	31.5	0.91-2.01	2.8	3.2	2.1	1.4*
Grenoble, France, 1947	18	31.8	0.63-1.93	1.4*	1.4*	2.0	4.9
St. Petersburg, Florida	9	36.0	0.58-1.15	2.2	2.5	2.0*	3.1
Victoria aq. Siphon 1	9	42.0	0.31-0.89	2.2*	2.3	2.4	4.3
Midland, Michigan	5	48.0	0.49-1.55	2.4	2.2*	4.7	7.1
Laboratory pipe, England	14	12.0	0.41-3.00	2.9	2.7*	3.2	8.2
Grenoble, France, 1947	15	31.4	0.80-1.95	2.8	2.5	2.3*	3.6
Chicago, Illinois	5	36.0	0.84-1.56	2.3	2.4	1.9	1.8*
St. Anthony Falls Lab	29	36.0	0.24-6.61	1.4*	3.5	3.0	13.4
Sta. Gertrudi's siphon	9	54.0	0.94-1.91	1.9*	2.0	2.0	3.4
Temecula siphon	9	48.0	1.19-2.42	3.7	3.8	3.3	1.9*
Rainbow siphon, part	12	54.0	0.55-1.92	3.9			2.9*
San Luis Rey siphon, part	10	48.0	0.70-2.43	1.6*	2.0	1.9	4.2
Escondido siphon	11	48.0	0.69-2.42	1.2*	1.8	1.8	4.1
Poway Valley siphon	11	48.0	0.69-2.41	1.9*			2.0
Minimum Mean Deviation				1.2	1.4	1.8	1.4
Maximum Mean Deviation				3.9	3.8	4.7	13.4
Average Mean Deviation				2.3	2.5	2.5	4.4

(based on Swanson and Reed, 1963)

* The formula for the best fit.

produces an effect on the velocity and head loss of ± 1.5 percent and $\pm$ 3 percent, respectively. Those differences were described as overshadowed by other uncertainties.

Values for the Hazen-Williams C-factor range from 140 for smooth and new pipes to 100 and 120 for old pipes in good condition, to 80 and as low as 40 for old pipes with tuberculation or heavy scaling. Table 2-12 summarizes design values commonly used.

Lamont (1981) compared the Hazen-Williams equation to the Darcy-Weisbach equation. Obtaining a correlation C-factor to Darcy friction factor C_f it was possible to plot on the Moody diagram lines for different C-factors, as shown on Figure 2-4. The effect of the diameter variation was considered very small for each C-factor plotted, therefore, the lines plotted are for a fixed diameter (300 mm). Analyzing the plot represented in Figure 2-4, Lamont (1981) drew the following conclusions about C-factors:

- C=150 to 155 for smooth pipes and pipe diameter $\geq$ 300 mm;
- C=120 to 150 for smooth pipes and pipe diameter $\leq$ 300 mm, according to the pipe diameter;
- C=100 to 150 for flow in pipes in the transition zone, according to the pipe diameter, and;
- C below 100 for flow in rough pipe, being not recommended the use of the Hazen-Williams equation, although accuracy may be improved by correcting the C-factor according to velocity and diameter.

Table 2-5 presented C-factors suggested by Lamont (1981) for various pipe diameters at a velocity of 0.9 m/s. Based on the conclusions summarized above, Lamont (1981) suggested corrections for the C-factors presented in this table. The corrections are summarized in Table 2-13.

Walski (1984) also stated the suitability of the Hazen-Williams equation for smooth pipes, or for velocities close to the velocity that the C-factor was measured. Considering that smooth pipes are not the general rule in a water distribution system, Walski (1984) proposed an alteration of the Hazen-Williams equation leading to an equation for smooth flows (equation 2-14 in Table 2-3), and another for rough flows:

$$v = 0.86\, C_o^{0.925}\, S^{0.5}\, R^{0.58}\, v_o^{0.075} \qquad (2\text{-}20)$$

where:

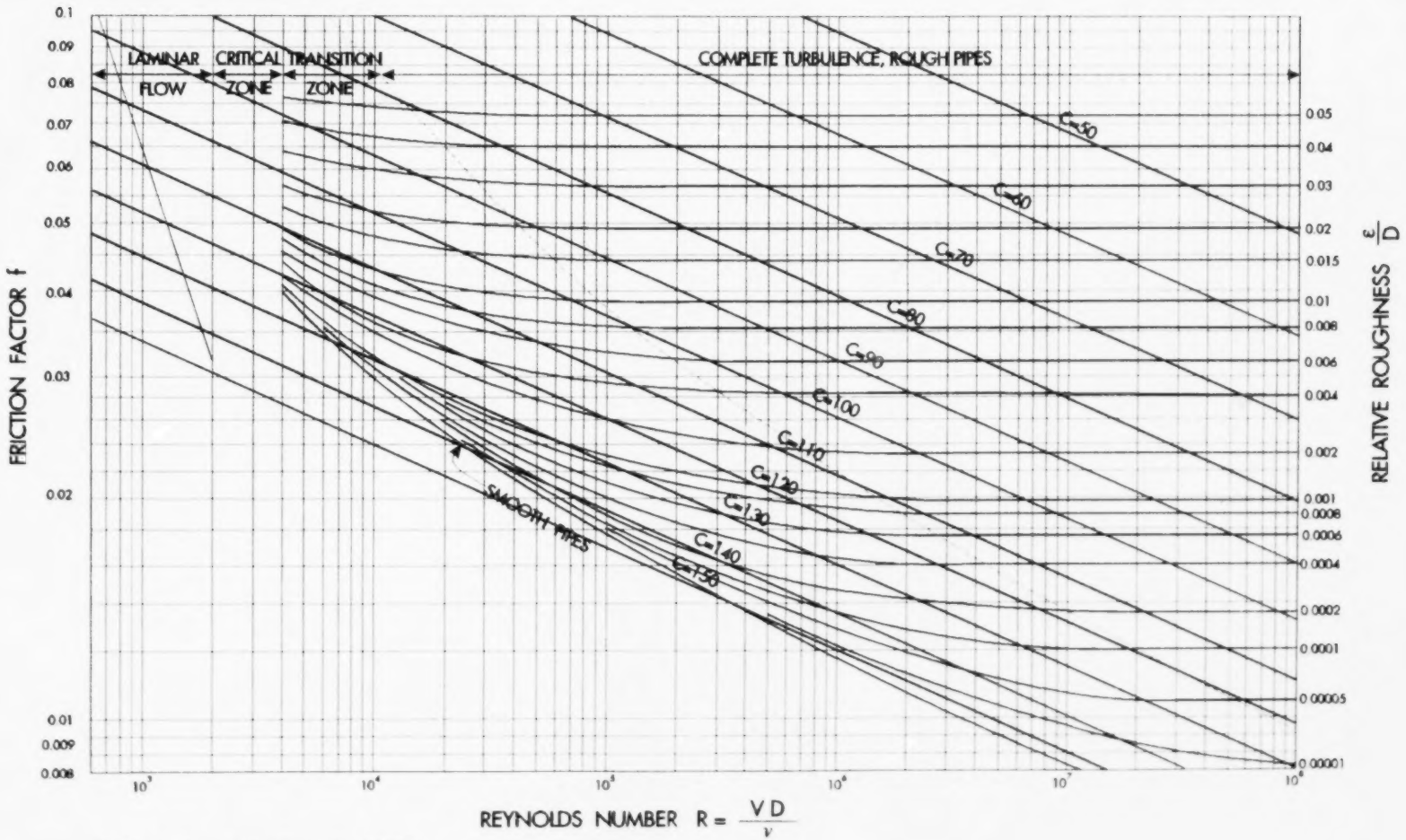
v = average velocity (m/s)
C_o = Hazen-Williams friction factor for a specific velocity v_o
S = hydraulic gradient
R = hydraulic radius (m)
v_o = initial velocity (m/s)

Table 2-12. Common Design Values of C-Factors

Type of Pipe	C Factor
Concrete Pressure Pipes (1)	
Diameters between 400 and 1200 mm	140
Diameters between 1350 and 2700 mm	145
Diameters 2850 mm and larger	150
Polivynylchloride (PVC) (2)	150
Steel Pipes (3)	
New pipes with cement or betuminous lining applied centrifugally	140
Old pipes with cement or betuminous lining applied centrifugally in good conditions or pipes lined in place	130
New pipes unlined	120
Old pipes unlined in good conditions	100
Old pipes unlined severely tuberculated	40 to 80
Cast Iron Pipes (3)	
New pipes with cement or betuminous lining applied centrifugally	140
Old pipes with cement or betuminous lining applied centrifugally in good conditions or pipes lined in place	130
New pipes unlined	120
Old pipes unlined in good conditions	100
Old pipes unlined severely tuberculated	40 to 80
Asbestos Cement Pipes (3)	140
Wood Stave Pipes (3)	120

(1) AWWA (1979)
(2) AWWA (1980a)
(3) ASCE (1975)

Figure 2-4. Plot of Hazen-Williams C-Factors in the Moody Diagram



(Based on Lamont, 1981)

Table 2-13. Correction for C-Factors at Velocities Different from 0.9 m/s

Values of C at 0.9 m/s (1)	Velocities Below 0.9 m/s for each HALVING or REHALVING of velocity relative to 0.9 m/s	Velocities Above 0.9 m/s for each DOUBLING or REDOUBLING of velocity relative to 0.9 m/s
C below 100	add 5 percent to C	subtract 5 percent from C
C from 100 to 130	add 3 percent to C	subtract 3 percent from C
C from 130 to 140	add 1 percent to C	subtract 1 percent from C
C above 140	subtract 1 percent from C	add 1 percent to C

(based on Lamont, 1981; Walski, 1984)

(1) C factors at velocity 0.9 m/s are presented on Table 2-5.

Walski (1984) suggested that the most accurate method to obtain C-factors is from field tests. Scobey (1966) had compiled tests in pipelines considered as smooth for more than 30 years. Similar work was done by Swanson and Reed (1963). Figures 2-5 to 2-14 illustrate the relation between C-factors obtained in those tests and velocity and diameter for precast concrete pipes, cement-lined steel and iron pipes, asbestos-cement pipes, and welded steel pipes. The data in these figures show a close agreement with the values suggested by Lamont (1981) for C-factors.

Summarizing, the Hazen-Williams equation is considered adequate for calculation of head loss in water distribution systems for the following reasons:

1. it is accurate in the ranges of water velocity and pipe diameters found in water distribution systems;
2. it is easy to use in head loss calculations;
3. it is easy to use in interactive calculation in water modelling;
4. friction factor (C) is easy to determine;
5. C-factor is well known by engineers involved in water distribution systems, and;
6. higher accuracy of C-factors may be obtained by applying a correction factor to the C-factor in function of pipe diameter and water velocity.

2.1.5. Field Measurement of Hazen-Williams C Factor

C factor is considered a weak point in the information required for modelling and designing a water distribution network (Eggener and Polkowski, 1976). Adopting a C factor based on the experience of an engineer is adequate as an initial guess (Walski, 1984). However, the analysis of the water distribution network using a model requires actual C factor values to obtain accurate results. Therefore, good field test sampling is very important for the development of network models and design (Hudson, 1954; AWWA, 1974; AWWA, 1989).

C factor measurements in small diameter pipelines ($\leq$ 300 mm) are developed by head loss tests using hydrants (AWWA, 1989). Those tests consist in measuring the head loss and flow between two consecutive hydrants in an isolated pipeline (Keith, 1946). This is achieved by measuring the heads in those two hydrants, and flushing and measuring flow in a third or more hydrants (AWWA, 1989).

The parameters required to develop head loss tests are length of the test section, diameter of the pipe tested, and flow and head loss in the test section (Walski, 1984; AWWA, 1989). Walski (1984) described methods and criteria for the measurement of the length and diameter of the tested pipe. Flow is usually measured using a hydrant Pitot tube (AWWA, 1976; AWWA, 1980b; Walski and Lutes, 1990). Friction head

Figure 2-5. C-Factors Based on Flow Tests in Precast Concrete Pipes
VELOCITY x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

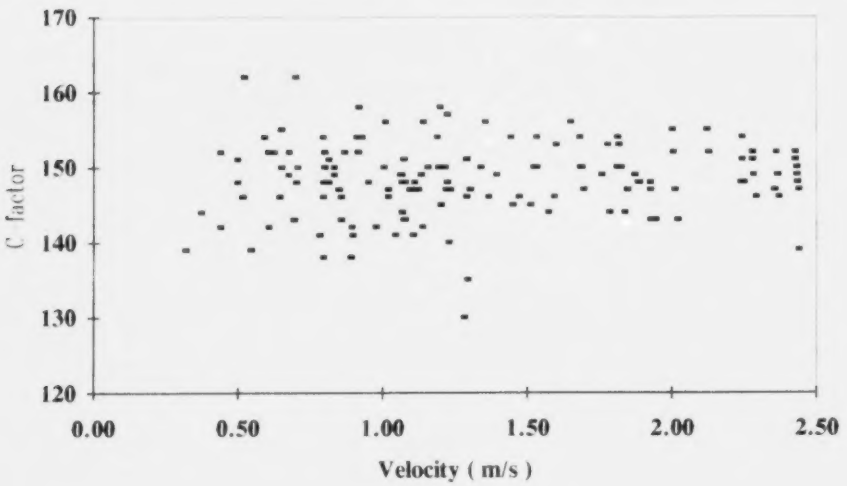


Figure 2-6. C-Factors Based on Flow Tests in Precast Concrete Pipes
DIAMETER x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

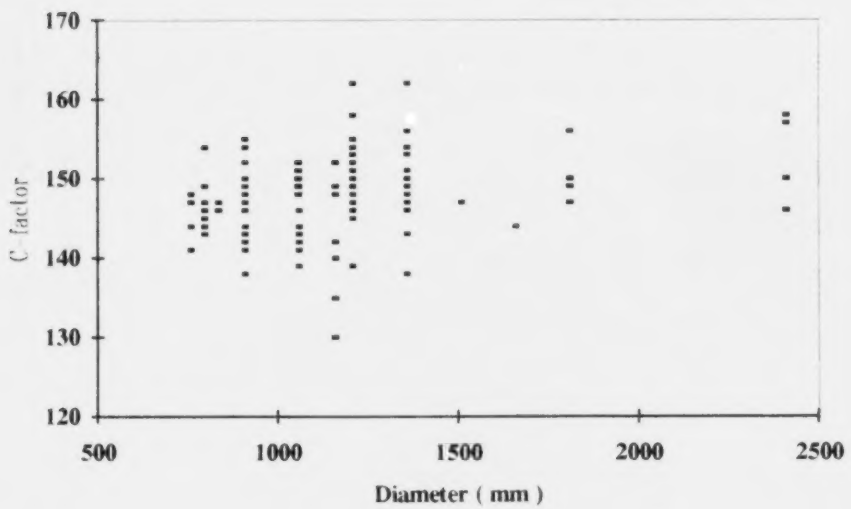


Figure 2-7. C-Factors Based on Flow Tests in Cement-Lined Iron and Steel Pipes
VELOCITY x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

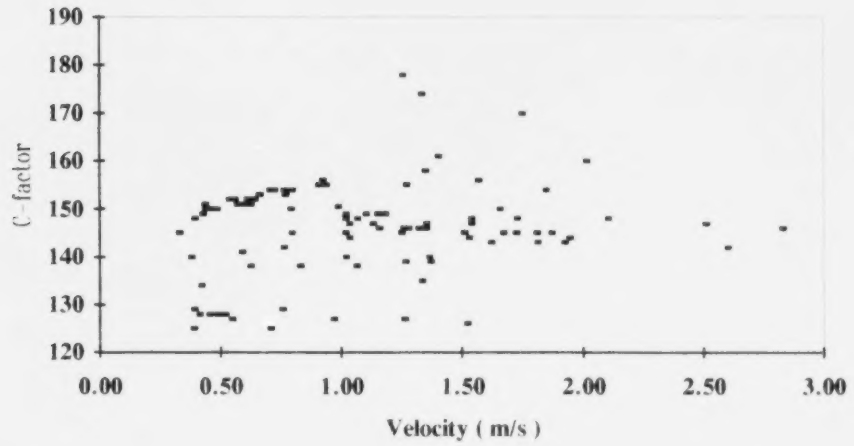


Figure 2-8. C-Factors Based on Flow Tests in Cement-Lined Iron and Steel Pipes
DIAMETER x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

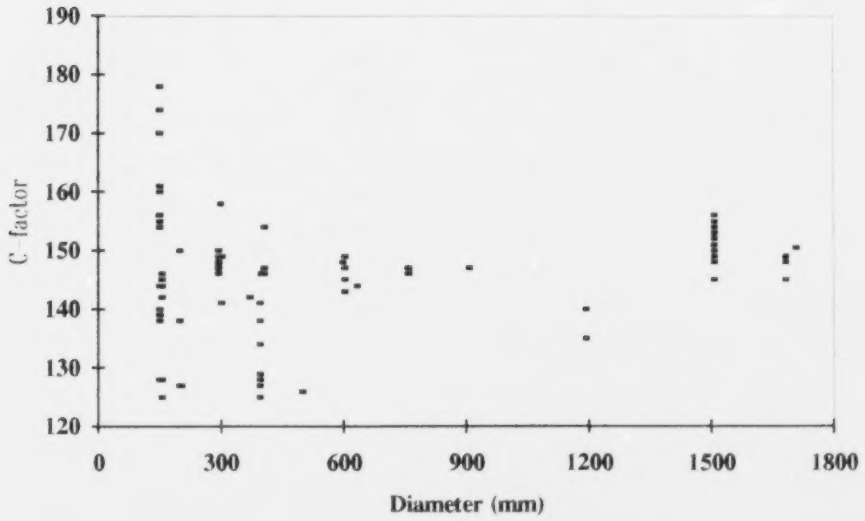


Figure 2-9. C-Factors Based on Flow Tests in Asbestos-Cement Pipes
VELOCITY x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

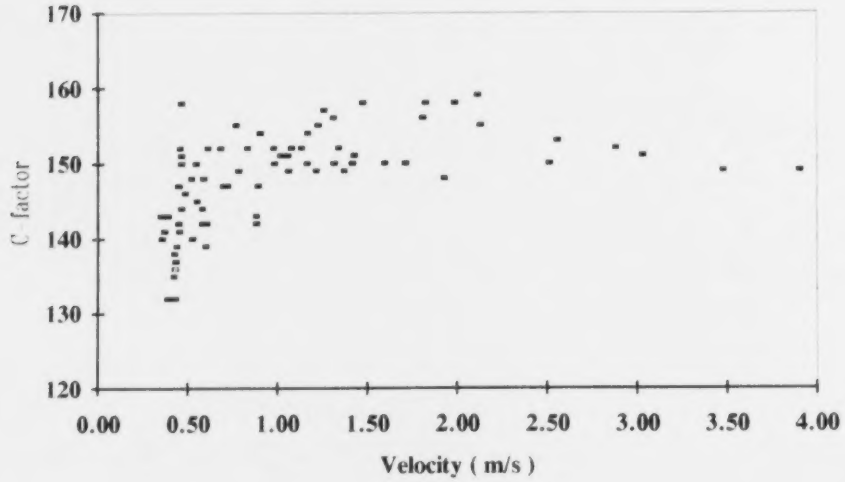


Figure 2-10. C-Factors Based on Flow Tests in Asbestos-Cement Pipes
DIAMETER x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

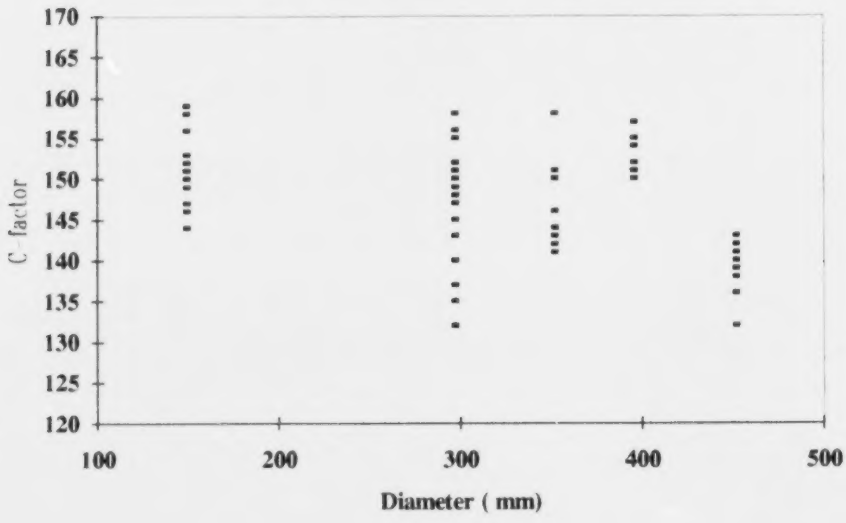


Figure 2-11. C-Factors Based on Flow Tests in Welded Steel Pipes
VELOCITY x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

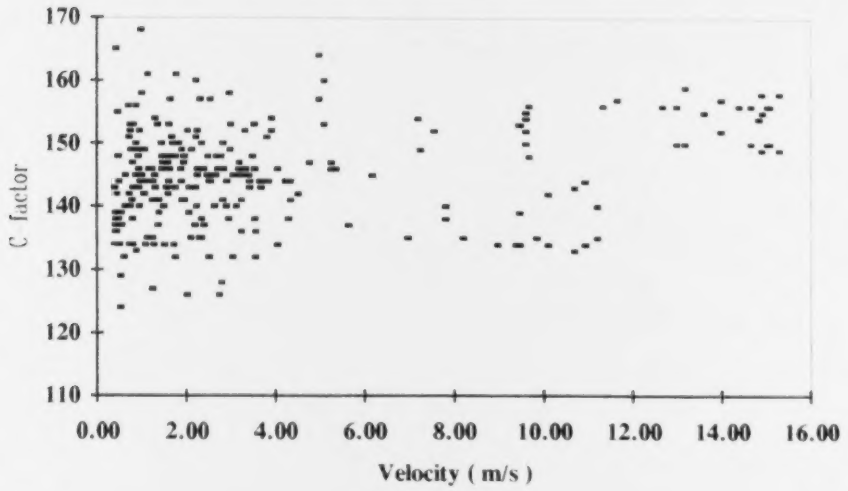


Figure 2-12. C-Factors Based on Flow Tests in Welded Steel Pipes
DIAMETER x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

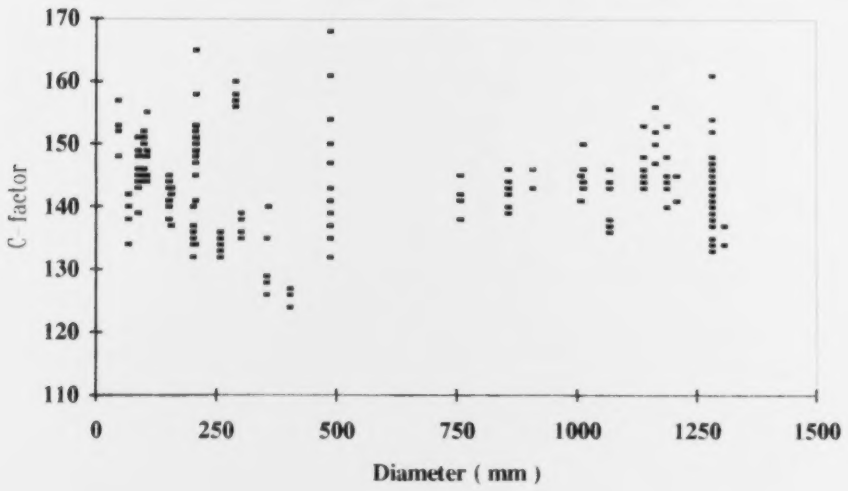


Figure 2-13. C-Factors Based on Flow Tests in Wrought Iron Pipes
VELOCITY x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)

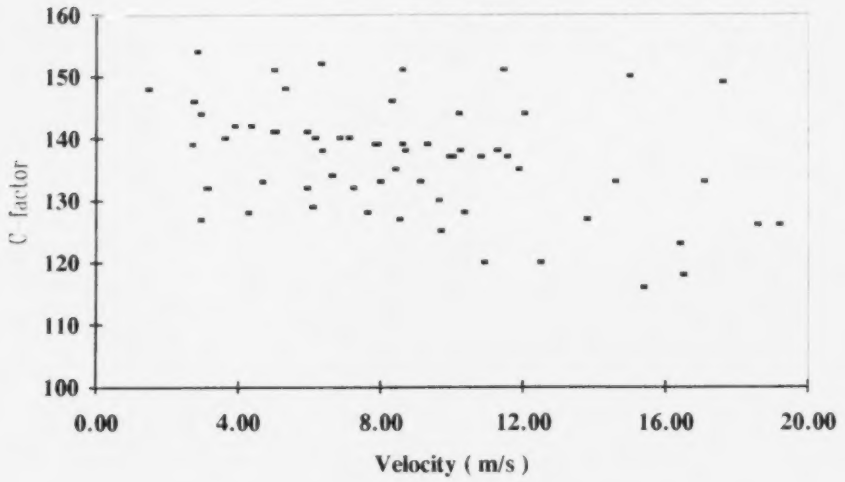
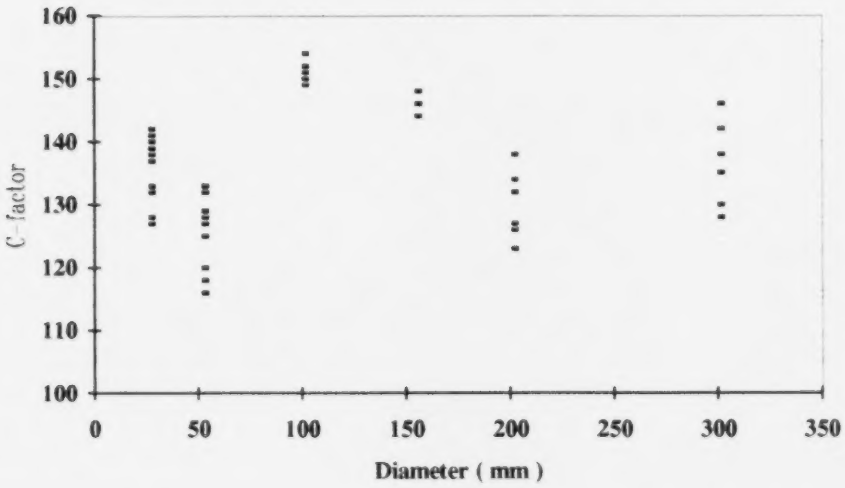


Figure 2-14. C-Factors Based on Flow Tests in Wrought Iron Pipes
DIAMETER x C-FACTOR
(based on data of Scobey, 1966; Swanson and Reed, 1963)



loss may be measured using two methods. In the first method, static pressures and elevations are measured at both hydrants at the end of the pipe test section (Keith, 1946; Hudson, 1954; Walski, 1984; AWWA, 1989). This method is called hydraulic grade line method and is illustrated in Figure 2-15. The second method does not require the elevation of the hydrants. It consists in laying a pipe parallel to the test section, connected to both hydrants at the ends of the test section. A differential pressure gage is installed in the parallel pipe and directly measures the head loss (Hudson, 1954; Walski, 1984; AWWA, 1989). This method is known as the parallel pipe method, shown schematically in Figure 2-16.

The C factor for the tested pipeline is obtained from the Hazen-Williams equation:

$$C = \frac{3.59\ Q}{D^{2.63}\left[\frac{h}{L}\right]^{0.54}} \qquad (2\text{-}21)$$

where:

C = Hazen-Williams friction factor
Q = flow rate (m^3/s)
D = diameter (m)
h = total head loss (m)
L = length of conduit (m)

The factors affecting head loss tests included:

1. Tested pipe characteristics (AWWA, 1989):
   - o Same diameter;
   - o Same age;
   - o Same material.
   - o As straight as possible.
2. Test section characteristics:
   - o No water loss (no open branches) (Hudson, 1954; AWWA, 1989);
   - o No negative pressures (Wyckoff, 1946);
   - o Necessary valve and fitting records and location to check effect of minor losses (Wyckoff, 1946);
   - o Flow in just one direction (Taylor, 1951);
   - o Necessary adequate length to generate measurable head losses (Taylor, 1951).

Figure 2-15. Head Loss Test for C-Factor Measurement
in Small Diameter Pipes
Hydraulic Grade Line Method
(based on Keith, 1946; Taylor, 1951)

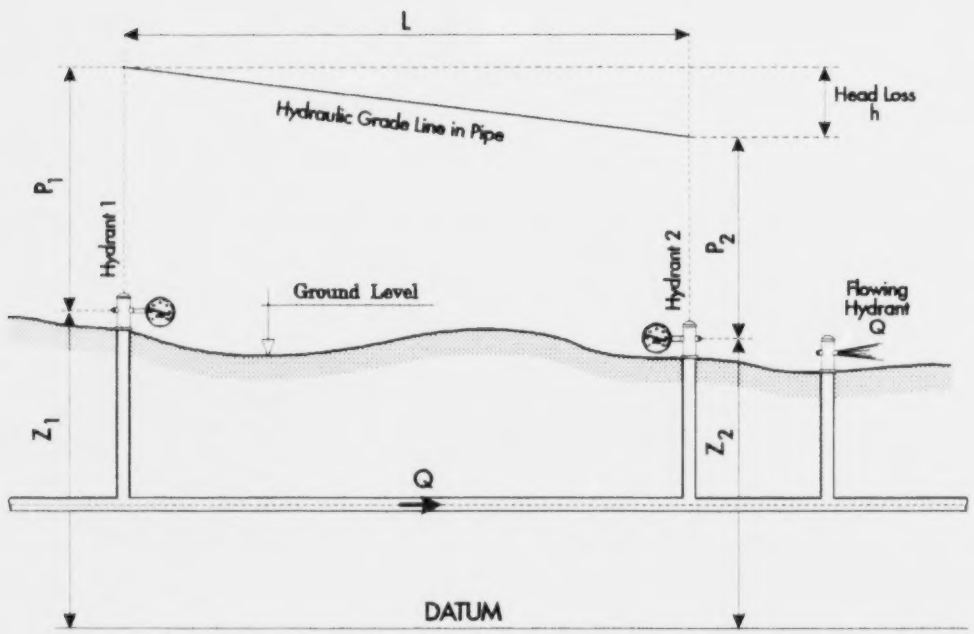
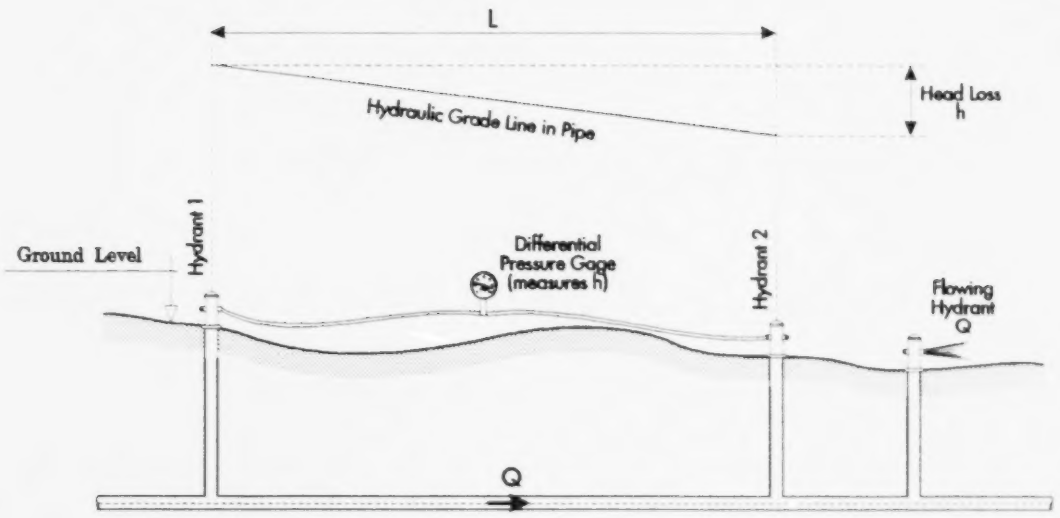


Figure 2-16. Head Loss Test for C-Factor Measurement
in Small Diameter Pipes
Parallel Pipe Method
(based on Walski, 1984)



3. Test development:

   - o Better results if test developed when minimum demand occurs (Keith, 1946; Taylor, 1951);
   - o Heads and flows readings taken simultaneously (Taylor, 1951).

Three methods can be used to measure C factors in large diameter mains. Two are based on the methods previously described for testing small diameter pipelines, as shown in Figures 2-17 and 2-18 (Keith, 1946; Taylor, 1951; Hudson, 1954; Walski, 1984; AWWA, 1989). The third is based on the velocity profile (Howard, 1967).

The velocity profile method was developed by Howard (1967), and is based on the theory of the boundary layer for flow in pipelines. The calculation of the C factor is obtained from the velocity profile from a pipe cross section such as the profile shown in Figure 2-19. The following procedures are then developed (Howard, 1967):

- o Plot each half of the velocity profile on a semi-logarithmic paper, obtaining the best fit line for each, as shown in Figure 2-20.
- o Calculate the shear velocity (v_*) based on the slope of each line, using the following equation:

$$\frac{v_{max} - v}{v_*} = \frac{-2.303}{K_K} \log\left[\frac{2y}{D}\right] \qquad (2\text{-}22)$$

where:

v_{max}	=	maximum velocity in the traverse (m/s)
v_*	=	shear velocity (m/s)
v	=	average velocity (m/s)
K_K	=	von Karman constant (defined as 0.4) (Howard, 1967)
y	=	distance from pipe wall (m)
D	=	diameter (m)

- o In equation 2-22, the slope of the line plotted on semi-logarithmic paper defines the term: ($2.303\ v_*/K_K$).
- o Calculate the wall shear stress using the following equation:

$$v_* = \sqrt{\frac{\tau_o}{\rho}} \qquad (2\text{-}23)$$

where:

τ_o = wall shear stress (kN/m^2)

ρ = fluid mass density (kg/m^3)

Figure 2-17. Hydraulic Grade Line Method for Head Loss Measurement in Large Diameter Mains

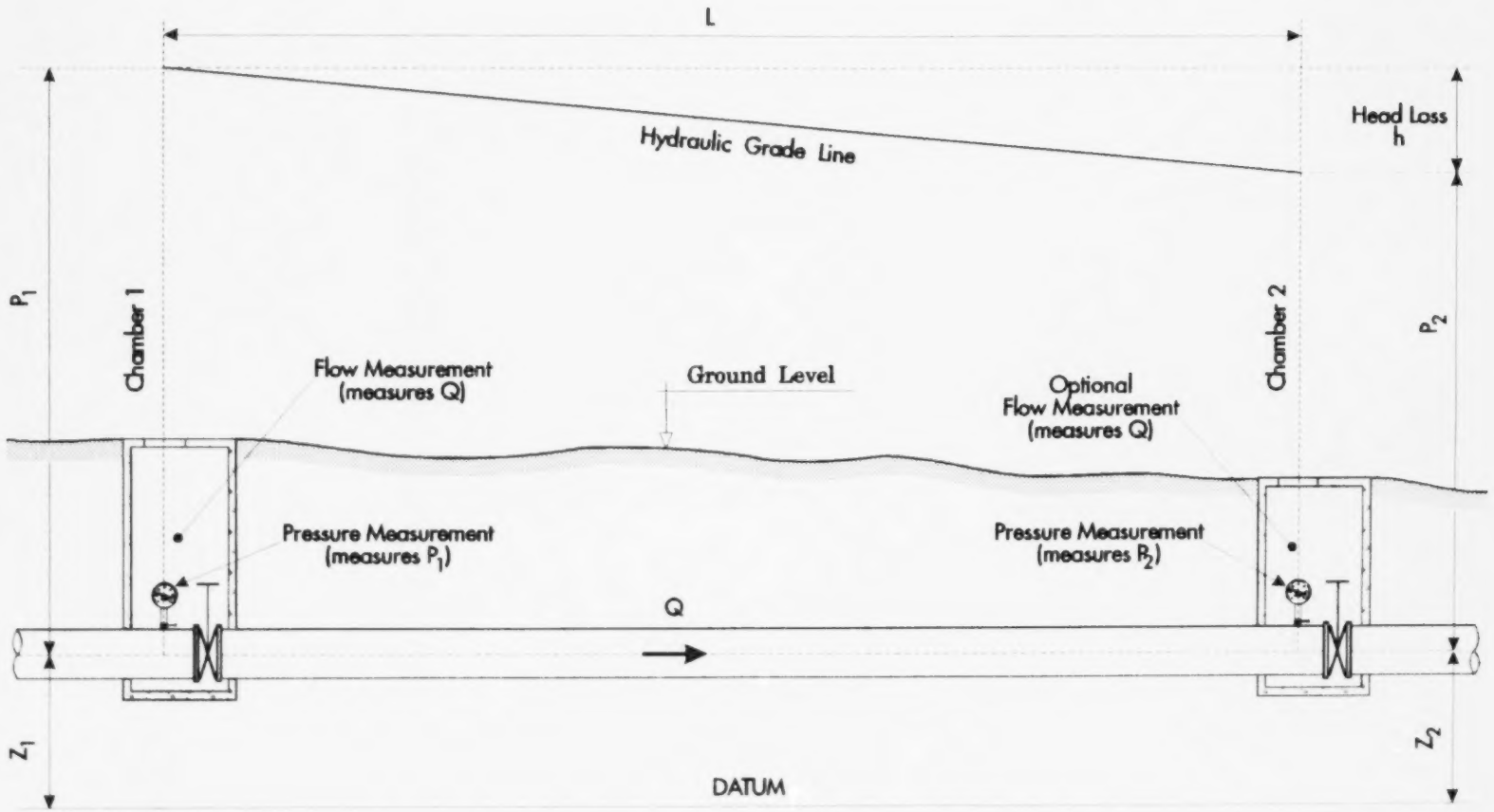


Figure 2-18. Parallel Pipe Method for Head Loss Measurement in Large Diameter Mains

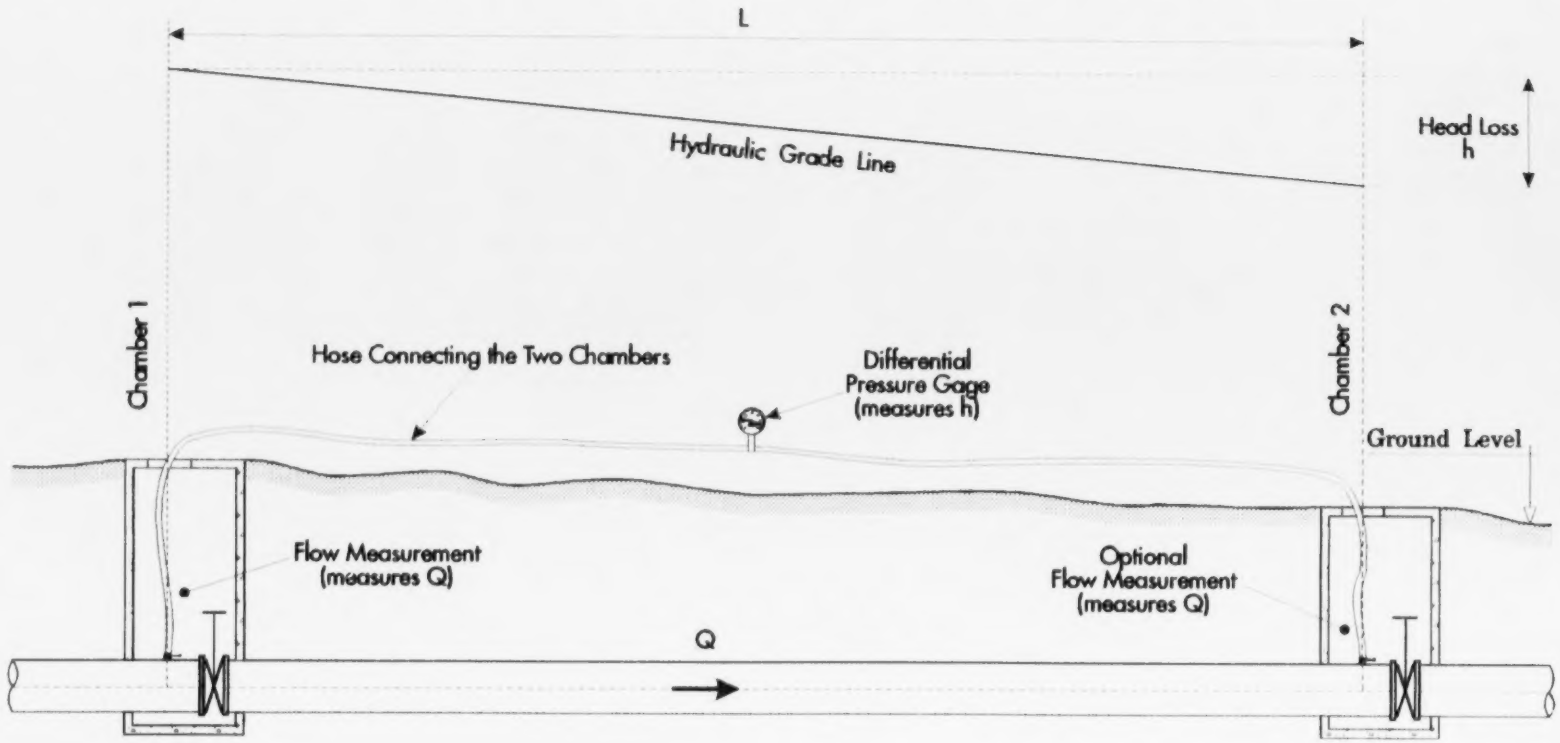


Figure 2-19. Velocity Profile Method for C-Factor Measurement in Large Diameter Mains - 10 Point Velocity Profile (based on Howard, 1967)

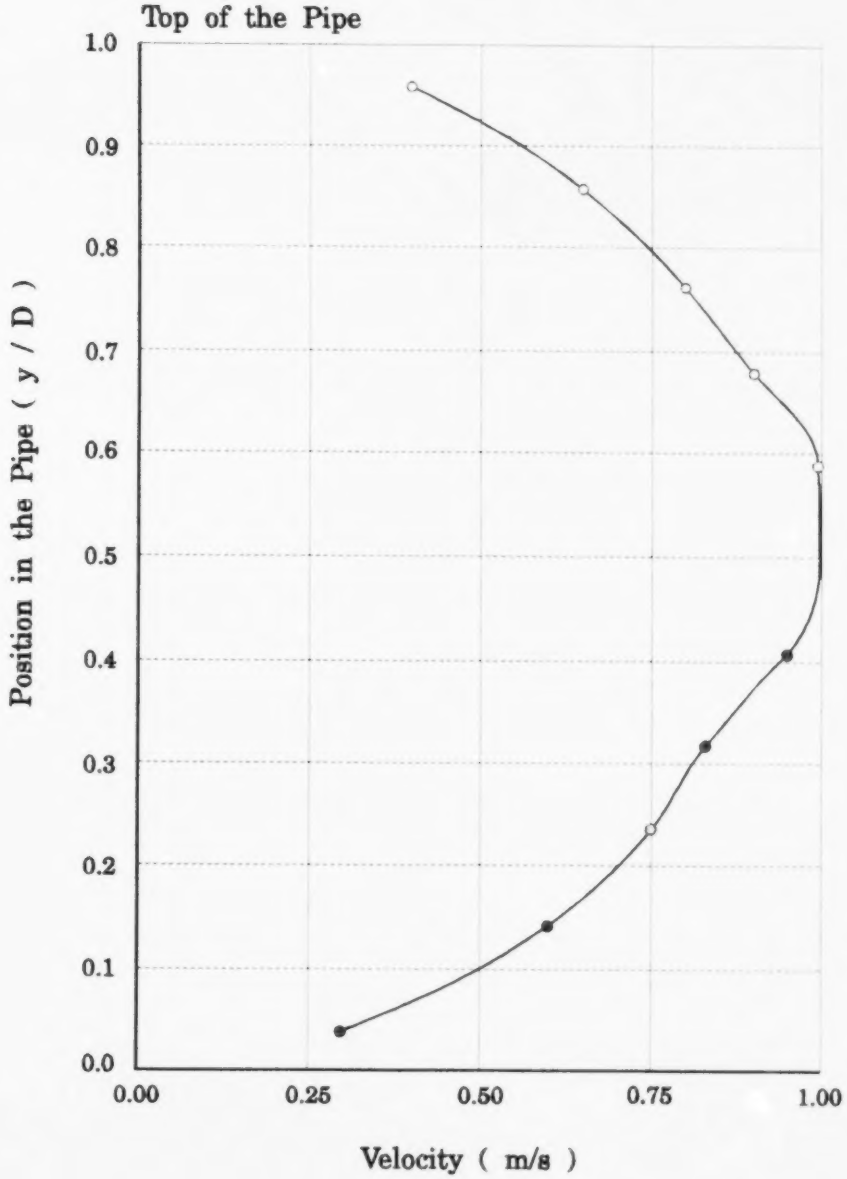
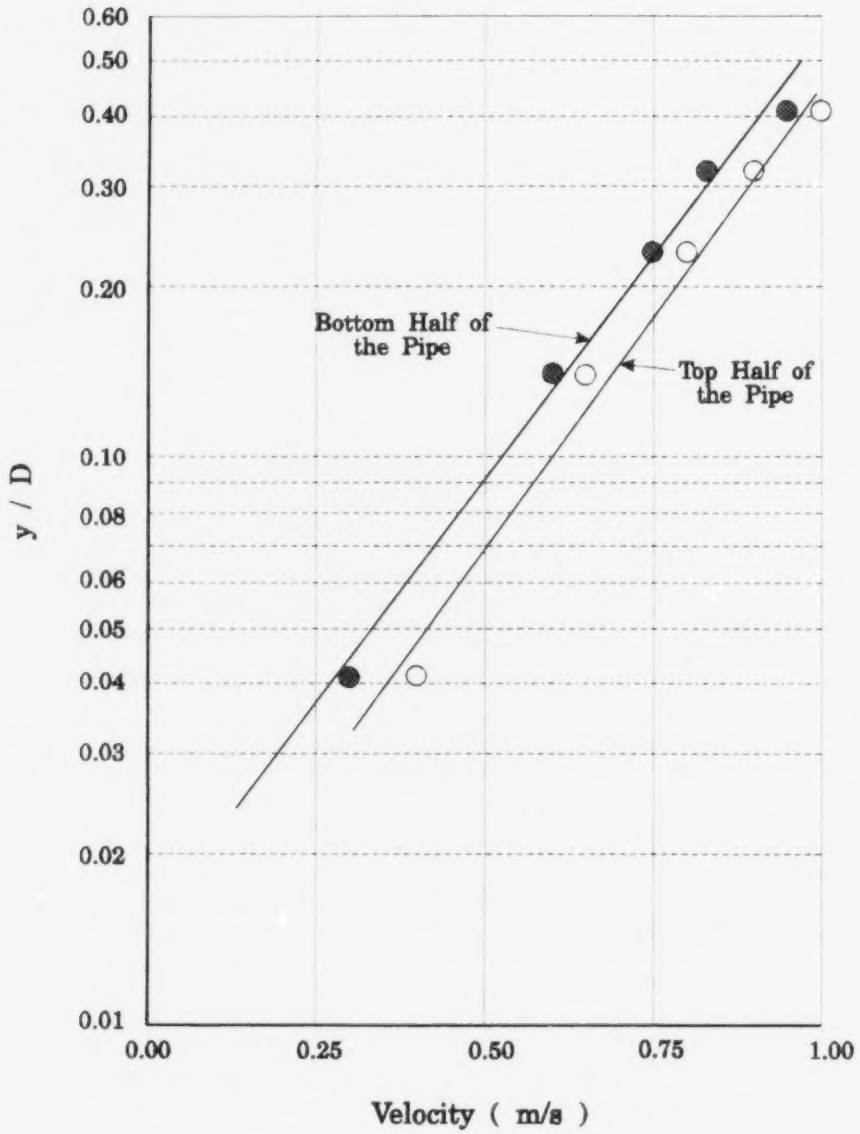


Figure 2-20. Velocity Profile Method for C-Factor Measurement in Large Diameter Mains - Semi Logarithmic Velocity Profile (based on Howard, 1967)



- The Hazen-Williams C-factor is calculated for each half of the velocity profile (C_T for the top half; C_B for the bottom half), based on the rearranged Hazen-Williams equation:

$$C = \frac{v}{0.849}\left[\frac{\rho\, g}{\tau_o}\right]^{0.54}\left[\frac{4}{D}\right]^{0.09} \qquad (2\text{-}24)$$

where:

C = Hazen-Williams friction factor

g = gravitational acceleration (m/s^2)

- The equivalent C factor (C_e) may be calculated for the section of the pipe based on the following equation:

$$C_e = \frac{1.46\ v}{\left[\left[\frac{v_T}{C_T}\right]^{1.85} + \left[\frac{v_B}{C_B}\right]^{1.85}\right]^{0.54}} \qquad (2\text{-}25)$$

where:

C_e = equivalent Hazen-Williams friction factor

C_T = Hazen-Williams friction factor for the top half of the pipe

C_B = Hazen-Williams friction factor for the bottom half of the pipe

v_T = average velocity in the half top of the pipe (m/s)

v_B = average velocity in the half bottom of the pipe (m/s)

Factors affecting the calculation of roughness by the velocity profile method are only the pipe diameter and the profile itself. Howard (1967) stated that the velocity profile method requires conditions in the field similar to those found in a laboratory. Therefore, good results are obtained only by cautiously locating test chambers in a position in which turbulence does not affect the velocity profile measurement.

Table 2-14 presents the principal advantages and disadvantages of the three methods that can be used for measuring C-factors in large diameter mains. The velocity profile method is theoretically the most accurate, but requires field conditions to measure the profile that are many times difficult to achieve. The two methods that measure the head loss in a section of pipe are based on the same principle. The parallel pipe method is more accurate because it measures directly the head loss, but it is more difficult to develop, requiring the installation of a parallel pipe. Finally, the hydraulic grade line method is the easiest to develop, just requiring two valve chambers tapped for pressure and flow measurement.

Table 2-14. Advantages and Disadvantages of the Methods to Measure C-Factor in Large Diameter Mains

Description	Grade Line Method	Parallel Line Method	Velocity Profile Method
Advantages			
Possible continuous monitoring using data-logger	Yes	Yes	No
Only one pressure measurement; not accumulate equipment errors	No	Yes	(1)
Not necessary pressure measurement	No	No	Yes
C factor is an average along the pipe section	Yes	Yes	No
All equipment protected inside valve chamber	Yes	No (2)	Yes (3)
Possible to use existing valve chambers	Yes	Yes	No
Calculations based on average velocity (just one value for each observation)	Yes	Yes	No
Process widely documented, tested, and accepted	Yes	Yes	No
Disadvantages			
Pressure measurement in both chambers (static pressure)	Yes	No	No
Necessary to measure length of pipe	Yes	Yes	No
Necessary measurement of pipe elevation at pressure measurement sites	Yes	No	No
Necessary isolation of branches in the network	Yes	Yes	No
Necessary parallel pipe for differential pressure measurement	No	Yes	No
Necessary construction of new chamber	No	No	Yes
Accuracy highly affected by proximity of valves and fittings	No	No	Yes
C factor dependent on the measurement of the velocity profile	No (4)	No (4)	Yes
Risk of vandalism	No	Yes	No
Disturb road traffic (parallel pipe crossing roads)	No	Yes	No
Necessary more than one flow measurement site to obtain C factor average	No	No	Yes

(1) Not necessary pressure measurement

(2) Only flow meter inside chamber, pressure measuring device is located outside chamber

(3) Flow meter inside chamber, but is necessary to construct special chamber

(4) Velocity profile necessary just once to find the position to measure the average velocity in the pipe

2.2. LOSS OF CARRYING CAPACITY IN WATER MAINS

2.2.1. General

Loss of carrying capacity in pipelines is mainly caused due to the reduction of the cross section of the pipe and due to the increasing roughness of the pipe walls (AWWA, 1962; AWWA, 1987b). Consequently, the reduction of the capacity of carrying water in a pipe results in lower C factors (Male and Walski, 1990). The most common factors that reduce carrying capacity of pipes are corrosion (Cruse et al, 1985; O'Day et al, 1986; Male and Walski, 1990), precipitation (Snoeyink and Jenkins, 1980; Walski, 1984), bacterial growth (AWWA, 1962; Swanson and Reed, 1963; Characklis, 1988).

Almost all of the above factors are a result of the physical, chemical and biological characteristics of the water (AWWA, 1962). These characteristics vary considerably from system to system, and usually, the quality of the treated water may even deteriorate inside the water distribution network (AWWA, 1987b).

Summarizing, systems that have high C factor reduction face an increase in operational costs. Indeed, the smaller the C factor, the higher the head necessary at the pump discharge to keep the same distribution conditions. Furthermore, a continuously deteriorating system requires much more attention with preventive maintenance (AWWA, 1987b).

2.2.2. Corrosion

Lamont (1981) described corrosion as a serious problem that affects mainly unlined cast iron and steel pipes carrying aggressive waters. Furthermore, corrosion may reduce the capacity of those pipes up to 70 percent after only 30 years of service.

Snoeyink and Jenkins (1980) defined corrosion as:

> "Corrosion is the deterioration of metallic structures, usually with the loss of metal to solution."

Corrosion occurs as a result of an oxidation-reduction reaction resulting from an aggressive water or from an electrolytic reaction promoted by an aggressive environment (AWWA, 1987b). Corrosion normally manifests in water pipelines in the forms of tuberculation, pitting, cracking and rusting (Snoeyink and Jenkins, 1980).

Cruse et al (1985) and O'Day et al (1986) described the principal factors responsible for internal corrosion as:

1. increases corrosion:
   - o low water pH;
   - o dissolved oxygen;

- o insufficient alkalinity (low buffering capacity);
- o elevated concentration of Total Dissolved Solids (TDS);
- o elevated temperature;
- o elevated flow rates;
- o chlorine;
- o soft water;
- o presence of iron in water (increase size of tubercles);
- o elevated ratio of halogen and sulfate to alkalinity (pitting), and;
- o growth of iron bacteria.

2. reduces corrosion:
   - o silica protective film;
   - o calcium protective film;
   - o phosphates (orto- and polyphosphates) protective films or by sequestering cations such as ferric ions;
   - o hard water.

Some strategies are adopted for corrosion reduction such as pipe lining and application of corrosion inhibitors (Cruse et al, 1985). Pipe linings are generally developed with coal-tar enamel, coal-tar epoxy, fusion-bonded epoxy, and cement-mortar (AWWA, 1987b). Corrosion inhibitors consist of chemically conditioning water to develop a thin layer of a non-corrosive substance to protect the pipe wall (Cruse et al, 1985; AWWA, 1986a; Male and Walski, 1990). The most common manner of achieving a protective film on the pipe walls is by allowing a deposition of calcium carbonate (AWWA, 1978). Other chemicals also used are inorganic phosphates and silicates (AWWA, 1986a; Male and Walski, 1990).

2.2.3. Precipitation

Precipitation occurs due to oversaturation of compounds dissolved in the water. The most common is calcium carbonate (Male and Walski, 1990), which is the predominant ion present in hard waters (Sawyer and McCarty, 1978). Depending on the characteristics of the treated water, dissolved ions may deposit on pipe walls.

Although not so frequent, Swanson and Reed (1963) described the precipitation of manganese oxides on pipe walls. Extremely small quantities of manganese are sufficient to considerably reduce the pipe carrying capacity. Other cases noticed are post-precipitation of alum and lime from the water treatment processes (AWWA, 1987b).

2.2.4. Biological Growth

Biological growth or fouling in pipes is a factor caused by bacterial activity in water, resulting in the buildup of organic deposits on the interior of the pipe (AWWA, 1987b). According to Lamont (1981), biological growth may reduce the carrying capacity of new smooth pipes by 25 to 50 percent after 10 to 20 years of service.

Bacterial growth in treated water distribution systems occurs because of bacterial breakthrough (growth resulting from bacteria not eliminated by disinfection) (Characklis, 1988). However, it occurs more frequently in raw water pipelines (Walski, 1984). The form of the occurrence is usually in a soft, uneven, slimy layer covering the pipe walls called bacterial film or biofilm. In general, biofilm may be easily wiped off the pipe walls, however, water velocity alone is not enough to remove it. The thickness of the film generally varies from a few millimeters up to 10 mm, and sometimes more (AWWA, 1962).

Other types of bacteria are associated with tubercle formation in metallic pipes. Larson (1939) described the effect of bacterial growth in that type of pipes (Cruse et al, 1985). Bacteria cells also may immobilize in extracellular polymeric substances and sediment deposits (Characklis, 1988). This forms a layer that concentrates nutrients for the cells, and provide a very adsorptive surface for cations, particulate material, and other cells, therefore, adequate for the bacterial growth.

Many people are mislead considering the bacterial growth capability in flow reduction because the biofilm has a smooth and slick appearance. However, this film consists of several tiny filaments that increase the surface area in the flow stream. Furthermore, the biofilm has an undulating pattern, generating turbulence in the pipe, consequently, increasing head losses (Swanson and Reed, 1963).

2.3. EQUIPMENT FOR PRESSURE AND FLOW MEASUREMENTS

2.3.1. General

Walski (1984) summarized the four parameters necessary to determine C factors as the internal diameter of the pipe, the length of pipe for the test section, the head loss (based on pressure), and the average velocity or flow in the pipe. The first two parameters may be easily measured without special equipment. However, velocity or flow, and pressure measurements are not so straightforward. Those parameters require special equipment that may vary from case to case, depending on the needs related to accuracy,

where and how to measure the parameters, the environment where those parameters have to be measured, and many other factors.

2.3.2. Pressure Measurement

Walski, 1984, classified pressure measuring devices in five categories: Bourdon gages, diaphragm gages, electric transducers, manometers, and bellows gages. The first two devices are frequently used for the measurement of the called "gage pressures," which are pressures exceeding the atmospheric pressure. The two latter are normally used for the measurement of differential pressures. Transducers, however, may be found both for measuring gage pressures and differential pressures.

Bourdon gages or Bourdon tubes are the most commonly used devices for gage pressure measurement in water distribution systems, being relatively accurate, resistant, and inexpensive (Walski, 1984). This type of pressure gages consists of a spring that suffers deformation by the action of pressure. This deformation is then transmitted to a rotating needle that being properly calibrated by the manufacturer shows the pressure on a dial (KSB, 1975).

Diaphragm gages are very similar to Bourdon gages. However, the action of the pressure is transmitted to the needle in the dial by the deformation of a diaphragm instead of a spring (KSB, 1975).

The basic form of a manometer consists of a piece of transparent U-shaped pipe connecting two points where the differential pressure is measured (Brater and King, 1976). The manometric fluid may be heavier than water when used for measuring large differential pressures, or lighter than water when the differential pressures are small (Walski, 1984). The differential pressure is:

$$\Delta p = \Delta_h (\gamma_w - \gamma_m) \qquad (2\text{-}26)$$

where:

Δp = differential pressure (Pa)

Δ_h = difference between height of columns of manometric fluid (m)

γ_w = specific weight of water (N/m^3)

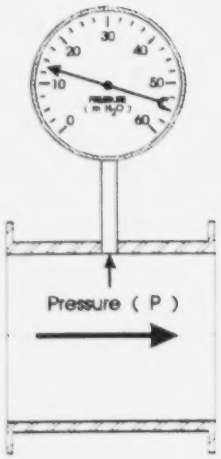
γ_m = specific weight of manometric indicator fluid (N/m^3)

Waslki (1984) described electric pressure transducers as one of the most modern devices for pressure measurement, although quite expensive when compared to other types of pressure gages. Moreover, the transducer alone is not useful, requiring a set of equipment for its adequate operation such as demodulators, recorders, indicators, amplifiers, cable, and transmitters. Transducers emit an electric signal as a voltage pulsation, which may be stored in a data recorder, or demodulated and shown on a digital indicator (Druck, 1984).

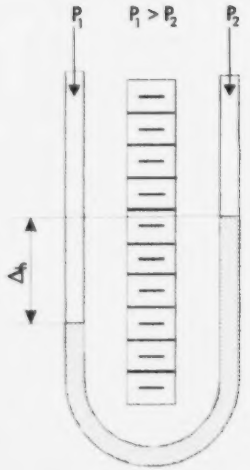
Figure 2-21 illustrates schematics of the most common pressure measurement devices.

Figure 2-21. Schematic of a Bourdon Gage, a Fluid Operating Manometer, and a Pressure Transducer

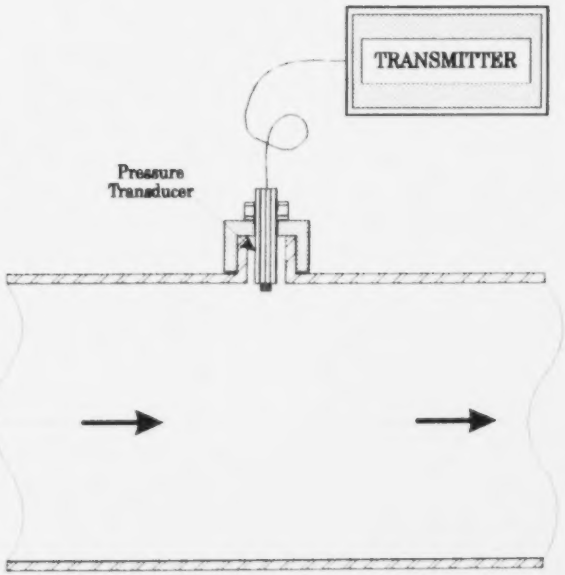
a. Bourdon Gage



b. Fluid Operating Manometer



c. Pressure Transducer



2.3.3. Flow Measurement

OMEGA Engineering (1988) classified flowmeters in four types: differential pressure, velocity, positive displacement, and mass meters. Only the first two are important for flow measurement in water distribution systems. Table 2-15 presents a summary of the most common types of flowmeters normally used for flow measurement in water works.

2.4. WATER DISTRIBUTION NETWORK MODELING

AWWA (1989) described the several areas in which computer modeling is used for water distribution network analysis. These areas are:

1. Planning.
    - o Long-range capital-improvement plans (scheduling, staging, sizing, preliminary routing and locating facilities).
    - o Main rehabilitation plans.
    - o System improvement plans.
    - o Main-sizing guidelines.
    - o Standard grid to provide adequate pressures.
2. Design
    - o Sizing facilities (pipelines, pumping stations, reservoirs, and valves for line, control, and pressure regulating).
    - o Fire-flow capability studies.
3. Operation
    - o Personnel training.
    - o Operating strategies.
    - o System troubleshooting (definition of the cause of problems).

The modelling process consists of the following steps: data collection, network solution based on the head loss equations, model calibration, and network analysis (Cesario and Davis, 1984; AWWA, 1989). If the process is developed using an unrealistic or uncalibrated model, misleading results may be obtained (Cesario, 1991). Therefore, actual values obtained in field tests represent an important role in network modelling. Moreover, Eggener and Polkowski (1976) described the friction factor of the pipes as the main point of concern in the input data in a model.

Table 2-15. Summary of Flow Measuring Devices for Use in Water Syatems

Flowmeter Element	Recommended Service	Rangeability Max:Min	Pressure Loss	Typical Accuracy	Scale Characteristics	Viscosity Effect	Upstream Length (Diam	Relative Cost
Differential Pressure Meters								
Orifice plate	Clean and dirty liquids; some slurries	4 : 1	Medium	2 to 4 full scale	Square root	High	10 - 30	Low
Wedge orifice plate	Slurries;viscous liquids	3 : 1	Low to Medium	0.5 to 2 full scale	Square root	Low	10 - 30	High
Flow nozzle	Clean and dirty liquids	4 : 1	Medium	1 to 2 full scale	Square root	High	10 - 30	Medium
Venturi tube	Clean; dirty and viscous liquids; some slurries	4 : 1	Low	1 full scale	Square root	High	5 to 20	Medium to High
Flow tubes		4 : 1	Very Low	1 full scale	Square root		10 - 30	Medium
Elbow meter	Clean and dirty liquids; some slurries	3 : 1	Very Low	5 to 10 full scale	Square root	Low	30	Low
Pitot rod	Clean liquids	3 : 1	Very Low	3 to 5 full scale	Square root	Low	20 - 30	Low
Target Meter	Clean; dirty and viscous liquids; some slurries	10 : 1	Medium	1 to 5 full scale		Medium	20 - 30	Medium
Rotameters	Clean, dirty and viscous liquids	10 : 1	Medium	1 to 10 full scale	Linear	Medium	None	Medium to Low
Velocity Meters								
Turbine meter	Clean, viscous liquids	20 : 1	High	0.25 of rate	Linear	High	5 - 10	High
Propeler meter	Clean, viscous liquids	10 : 1		2 to 5 rate	Linear		5	Medium to High
Electromagnetic	Clean, dirty, viscous conductive liquids	40 to 1	None	0.5 of rate	Linear	None	5	High
Ultrasonic - time-of-travel	Clean, viscous liquids	20 : 1	None	1 to 5 of full scale	Linear	None	5 to 30	High
Ultrasonic -Doppler	Dirty, viscous liquids and slurries	10 : 1	None	5 of full scale	Linear	None	5 to 30	High
Vortex	Clean and dirty liquids	10 : 1	Medium	1 of full scale	Linear	Medium	10 to 20	High

(based on Hayward, 1979; De Carlo, 1984; Walski, 1984; OMEGA, 1988)

2.5. SUMMARY

The Hazen-Williams equation is not perfect theoretically, but undoubtedly it is the most used equation in North America (Sharp and Walski, 1988). Walski (1984) noted that:

> "While the Darcy-Weisbach equation is more theoretically correct and can be applied to virtually any fluid, the Hazen-Williams equation does not require reevaluation of the friction factor at every iteration. Since engineers have a good feel for the value of the Hazen-Williams C factor over a wide range of pipe sizes, types and ages, the Hazen-Williams equation can be used with acceptable error, especially for hydraulically smooth pipes."

Many researchers have justified the use of the Hazen-Williams equation in water distribution system calculations (Hinds, 1946; Swanson and Reed, 1963; Lamont, 1981; Sharp and Walski, 1988). Therefore, the Hazen-Williams equation will be used in this study.

AWWA (1974) suggested the development of good field sampling for C factors in water mains, reducing the major source of errors in network simulations and main designs. Moreover, Eggener and Polkowski (1976) defined the C factor as: "The weakest piece of input information used in modelling . . . " Consequently, obtaining actual C factors in various pipelines for a wide range of flows, avoids incurring in wrong pipe roughness estimate (Walski, 1986). In summary, correct roughness values allow the knowledge of the actual system capabilities, helping the design, operation, and planning engineers to better evaluate their decisions. Therefore, the results of this study will help the City of Edmonton to improve the performance of the simulation model currently used for planning and design.

3. MATERIALS AND METHODS

3.1. WATER DISTRIBUTION SYSTEM IN THE CITY OF EDMONTON

3.1.1. Overview

The water distribution system of the City of Edmonton is responsible for supplying water for a population of more than 750,000 people, not only in Edmonton, but also in neighboring municipalities such as Leduc, Beaumont, Calmar, Spruce Grove, Stony Plain, Saint Albert, Morinville, Namao, Fort Saskatchewan, Gibbons, Redwater, Bruderheim, Lamont, Chipman, Mundare, Vegreville, and Sherwood Park (City of Edmonton, 1989; 1991).

Water produced in the system is taken from the North Saskatchewan River and treated at two water treatment plants, which have a normal combined production of 430 ML/d. Each water treatment plant has a high lift pumping station that feeds directly the water distribution network, which consists of approximately 2,700 km of pipes with diameters ranging from 100 mm to 1,350 mm. The total storage capacity of the system is 653 ML, consisting of two reservoirs located at each water treatment plant and eleven other ones connected to the water distribution network. Each reservoir has a pumping station, which helps to keep a uniform level of pressure and water supply during periods of higher demand and emergencies (City of Edmonton, 1987; 1991).

Figure 3-1 shows the location of the main components in the water distribution system of the City of Edmonton, including water treatment plants, reservoirs, and the location the regional customers that receive water from Edmonton.

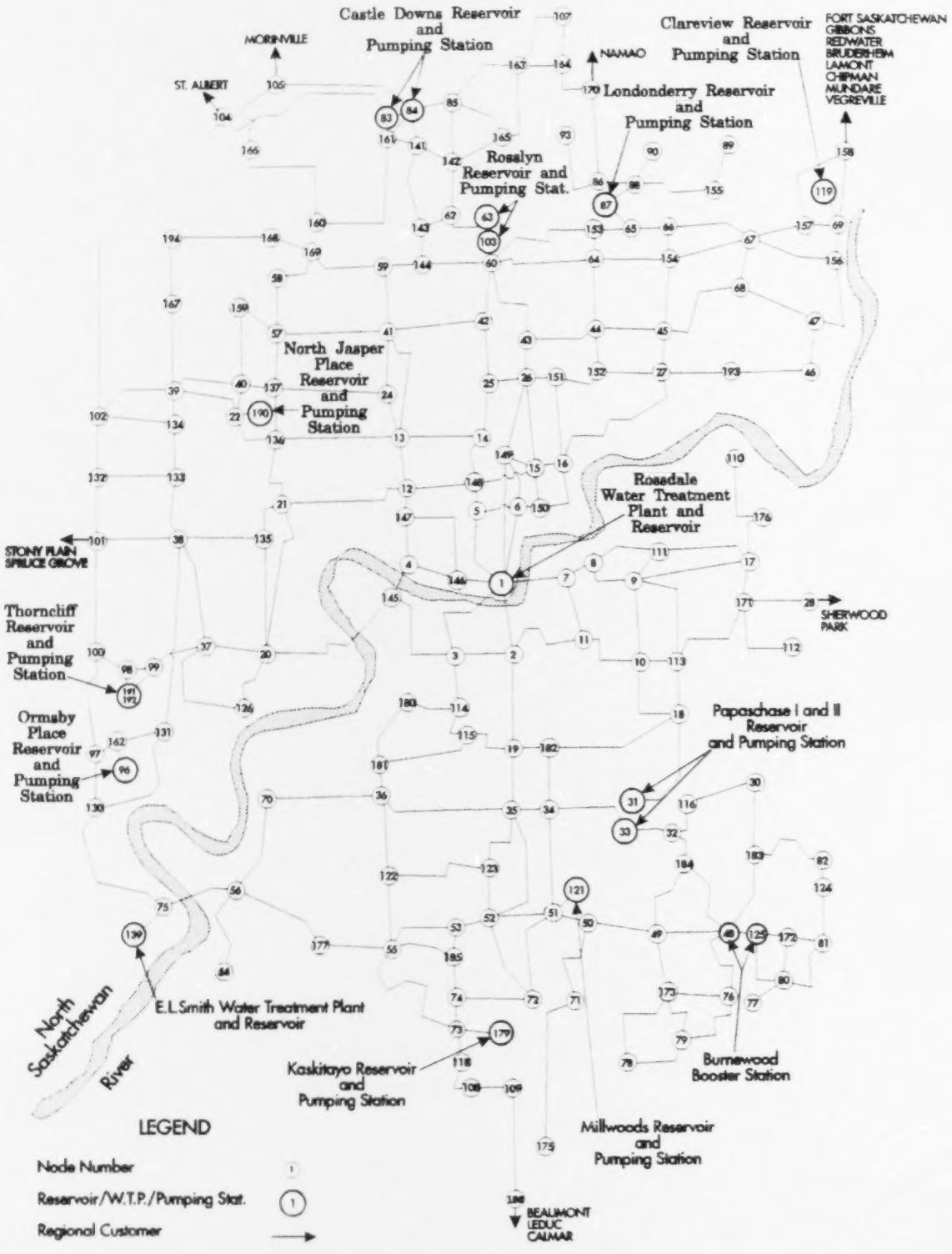
3.1.2. Water Treatment Plants

The oldest water treatment plant is Rossdale, located close to the Edmonton's downtown area. The second plant, called E. L. Smith, was constructed in 1975 and expanded in 1983, and is located upstream the City center. The annual production of each plant is approximately 60,000 ML (City of Edmonton, 1987; 1989; 1991).

The water treatment process employed in both plants consists of the following units (City of Edmonton, 1987; 1989):

- Clarification: clarification is achieved in horizontal and vertical flow settling tanks, using as flocculation agent aluminum sulfate.
- Softening: water taken from the North Saskatchewan river is naturally hard, requiring softening, which is achieved using slaked lime.
- Filtration: the remaining suspended solids are removed in sand and anthracite double layer filters.

Figure 3-1. Schematic of Edmonton's Water Distribution System



- o Disinfection: water disinfection may be obtained applying chlorine, chloramines, or chlorine dioxide. Recently the City has used almost exclusively the latter.

- o Taste/odor: during the spring, when run-offs increase the presence of organic compounds in water, powdered activated carbon is used for taste and odor control.

- o pH control: during the softening process, the treated water has its pH raised, to control the final pH of the water, recarbonation using carbon dioxide is used.

- o Fluoridation: hydrofluorsilicic acid is applied to the treated water.

3.1.3. Reservoirs and Pumping Stations

Table 3-1 presents a summary of the storage capacity of the reservoirs in the City of Edmonton. Storage volume in each reservoir corresponds to:

- o Gross storage (volume of reservoir completely full);

- o Operating storage (available storage + fire storage);

- o Fire storage (fire emergencies);

- o Available storage (hourly demand peaks + daily demand peaks).

Table 3-2 shows a summary of the pumping facilities for each reservoir in the system.

Table 3-3 gives a summary of the operating pressure limits for each pumping stations in the system.

3.1.4. Water Distribution Network

The water distribution network in the City of Edmonton, excluding the networks of the Regional customers, totals 2,700 km of pipelines. This system includes pipes from 100 mm to 1,350 mm manufactured with several materials such as asbestos cement, unlined cast iron and steel, polyvinylchloride (PVC), pressure concrete pipes, reinforced fiberglass, lined ductile iron, and wood stave (City of Edmonton, 1991).

Table 3-4 presents a summary of the total extension of pipes in Edmonton's water distribution network until the end of 1990.

Table 3-1. Storage Capacities of the Existing Reservoirs
(situation in December 1990)

Reservoir or Water Treatment Plant	Storage (ML)			
	Gross	Operating	Fire	Available
Rosslyn	123.08	110.05	20.22	89.83
Papaschase I	57.11	55.68	9.98	45.70
Papaschase II	25.03	20.83	4.23	16.60
Londonderry	45.24	41.68	2.67	39.01
Thorncliff	45.35	40.04	4.71	35.33
Mill Woods	56.23	52.90	7.13	45.77
North Jasper Place	46.06	34.40	7.50	26.90
Ormsby Place	45.27	40.40	4.81	35.59
Clareview	64.60	53.46	4.75	48.71
Castledowns	34.04	25.74	2.51	23.23
Kaskitayo	28.94	25.74	4.77	20.97
Sub-Total	570.95	500.92	73.28	427.64
Rossdale	100.00	100.00	0.00	100.00
E. L. Smith	137.47	125.60	0.00	125.60
Sub-Total	237.47	225.60	0.00	225.60
Total	808.42	726.52	73.28	653.24

(based on City of Edmonton, 1987; 1991)

Table 3-2. Pumping Facilities
(situation in December 1990)

Pumping Station	Year Built	Number of Pumps			Maximum Discharge Flow by Pump (ML/d)
		Total	Constant Speed	Variable Speed	
Rosslyn 1	1955	3	3	0	3 @ 20
Rosslyn 2	1969	2	2	0	3 @ 22
Rosslyn 3	1969	2	2	0	2 @ 26
Papaschase I	1963	2	2	0	2 @ 26
Papaschase II	1976/82	3	1	2	3 @ 16
Londonderry	1968/71	3	0	3	2 @ 13, 1 @ 23
Thorncliff	1970	3	3	0	3 @ 12
Mill Woods	1974/79	5	4	1	3 @ 20, 2 @ 24
North Jasper Place	1974	4	3	1	2 @ 13, 2 @ 26
Ormsby Place	1977	4	2	2	2 @ 13, 2 @ 24
Clareview	1979	3	3	0	1 @ 14, 2 @ 27
Castledowns	1979	3	1	2	3 @ 17
Kaskitayo	1980	3	2	1	3 @ 10
Clover Bar	N/A	3	3	0	N/A
Rossdale Plant *	1987	6	4	2	6 @ 90
E. L. Smith Plant *	1976	4	1	3	2 @ 90, 2 @ 180
Burnewood Booster St.	1985	3	2	1	3 @ 17
Total		56	38	18	

* High lift pumps

(based on City of Edmonton, 1987; 1991)

Table 3-3. Operating Pressure Limits

(situation in December 1990)

Facilities	Pressure Transmiter Elevation (m)		Operating Pressure Range kPa (M.T.H.)				
Treatment Plants	**Discharge**		**Minimum**	**Normal Range**			**High**
Rossdale Plant	622.25		840 (708)	860 (710)	880 (712)	900 (714)	930 (717)
E. L. Smith Plant	620.85		950 (718)	960 (719)	980 (721)	1000 (723)	1035 (726)
Reservoirs	**Inlet**	**Discharge**	**Fill**	**Alarm**	**Minimum**	**Normal**	**High**
Rosslyn 1		669.87		370 (708)	390 (710)	420 (713)	450 (716)
Rosslyn 2		671.42	300 (702)	360 (708)	380 (710)	410 (713)	440 (716)
Rosslyn 3		669.14	--	450 (715)	470 (717)	520 (722)	630 (733)
Papaschase I		690.99	variable	160 (707)	180 (709)	230 (711)	300 (714)
Papaschase II	689.06	690.42	70 (696)	380 (729)	450 (736)	450 (736)	490 (740)
Londonderry	677.91	678.21	210 (700)	385 (717)	460 (725)	490 (728)	540 (733)
Thorncliff	672.02	672.02	350 (708)	370 (710)	390 (712)	420 (715)	450 (718)
Mill Woods	678.82	678.83	270 (706)	540 (734)	575 (737)	585 (738)	600 (740)
North Jasper Place	675.12	675.12	290 (705)	340 (710)	370 (713)	390 (715)	430 (719)
Ormsby Place	679.41	679.41	325 (712)	345 (714)	365 (716)	380 (718)	420 (722)
Clareview	649.73	648.95	430 (693)	490 (700)	550 (706)	580 (709)	610 (712)
Castledowns	678.96	677.99	260 (705)	380 (717)	460 (725)	490 (728)	540 (733)
Kaskitayo	673.84	673.84	345 (709)	470 (722)	490 (724)	600 (735)	680 (743)
Booster		**Discharge**		**Alarm**	**Minimum**	**Normal**	**High**
Burnewood		695.05		550 (751)	580 (784)	580 (754)	675 (764)

(based on City of Edmonton, 1991)

Table 3-4. Transmission and Distribution Mains
(situation in December 1990)

Type of Pipe	Length (km)						
	< 150 mm	150-250 mm	300-400 mm	450-700 mm	750-1200 mm	> 1200 mm	Total
Cast Iron (unlined)	39.1	922.4	59.2	12.5			1033.1
Ductile Iron (lined)	0.1	3.2	0.2	0.1			3.6
Fiberglass Reinforced				3.4	3.6		7.0
Asbestos Cement	2.8	688.6	289.2	79.9			1060.5
Steel		11.2	24.8	45.9	35.4	1.1	118.3
Pressure Concrete Pipe				25.9	88.4	6.4	120.6
Wood Stave		0.3					0.3
PVC		304.9	51.6	0.4			356.9
Total	42.1	1930.4	424.9	168.0	127.3	7.4	2700.2

(based on City of Edmonton, 1991)

3.2. SELECTION OF TRANSMISSION MAINS TO BE TESTED

3.2.1. Selection Criteria

The criteria used for the selection of large diameter mains to be tested in Edmonton's water distribution network included the following:

1. Test pipe requirements:
   - o diameter $\geq$ 600 mm;
   - o same diameter along test section;
   - o same material along test section;
   - o same age along test section.
2. Test section requirements:
   - o length of test section:
     - ·· length $\geq$ 2,000 m for velocities $\leq$ 0.80 m/s;
     - ·· length $\geq$ 1,300 m for velocities $>$0.80 m/s.
   - o constant flow direction along the test section;
   - o test section as straight as possible;
   - o no negative pressures along test section;
   - o no water loss along the pipe test section (necessary to isolate the pipe test section closing all branches along the section):
     - ·· avoid the necessity of closing important branches;
     - ·· branches closed have to keep similar pressure and flow conditions, being fed by other pipes.
   - o number of valves and fittings in the test section not larger than the average number of valves and fittings in the system (avoid mains with excess of valves and fittings).
3. Valve chamber requirements:
   - o use only existing chambers (no chamber construction);

- necessary to have two chambers in each pipe test section:
  - one chamber for pressure measurement;
  - one chamber for pressure and flow measurement.
- chamber for pressure measurement:
  - at least one 50 mm air vent (gate or angle valve).
- chamber for pressure and flow measurement (more than one air vent):
  - pressure: at least one 50 mm air vent (gate or angle valve);
  - flow: at least one 50 mm air vent with gate valve.
- chamber for pressure and flow measurement (only one air vent):
  - pressure and flow: one 50 mm air vent with gate valve (using an auxiliary tee for measuring both parameters).
- chambers have to be large enough for equipment installation and operation:
  - pressure with gate valve air vent: at least 0.40 m between top of valve and chamber walls;
  - pressure with angle valve air vent: at least 0.30 m between top of valve and chamber walls;
  - flow with gate valve air vent: at least at least 0.90 m between top of gate valve and chamber walls;
  - pressure and flow in a single gate valve air vent: at least at least 1.30 m between top of gate valve and chamber walls.
- use chambers already prepared for flow measurement (Table 3-5 describes the characteristics of the 23 chambers prepared by the City of Edmonton Public Works for flow measurement. Figure 3-2 shows the location of those chambers);
- use chambers where is possible to change one angle valve for a gate valve (necessary to isolate pipe for approximately one hour);
- avoid chambers where is necessary to install a new tap in the pipe.

Table 3-5 - Chambers in Water Transmission Mains Already Prepared for Flow Measurement

No. of the Chamber (1)	Model Node to Node(2)	Pipe Test Section				Control Valve			Air Vent Valves			Flow Direction	Possible to Test	Observations
		Diameter (mm)	Material	Length (m)	Year	Diameter (mm)	Type	ID Number	Diameter (mm)	Type	Position to Control Valve			
1	53-55	900	Pres.Concrete	1505	1975	750	Gate	25269	50	Gate-45	E	W->E	Yes	Chamber 1 has one tap; length is short
2	56-177	1050	Steel	3542	1975	900	Gate	25345	50 50	Gate-45 Angle	W E	W->E	Yes	Very good for C-factor test
3	56-70	450	Asb.Cement	--	1978	450	Butterfly	29891	50 50	Ball Ball	N S	S->N	No	Gate valve for flowmeter not installed; Section with different materials
4	131-162	750	Pres.Concrete /Fiber Glass	--	1976	600	Gate	28366	50 50	Gate-45 Angle	E E	W->E	No	Different materials
5	97-130	1050	Pres.Concrete	--	1976	900	Butterfly	28521	50 50	Gate-45 Gate	N S	S->N	No	Downstream valve: Different materials Backstream valve: need to isolate reservoir and important branches
6	14-148	750	Welded Steel	1372 to 1700	1950	600	Gate	6252	50	Gate-45	S	S->N	Yes	Ch.6 has just one tap; length is short
7	4-146	1050	Pres.Concrete	1325	1958	750	Gate	11954	50 50	Gate Angle	E W	E->W	Yes	Length is very short
8	4-147	900	Lined Steel	1790	1958	750	Gate	11967	50 50 50	Gate-45 Angle Angle	S S N	S->N	Yes	Necessary to close many branches
9	4-145	750	Steel	2548	1960	750	Butterfly	9657	50 50	Gate-45 Angle	E W	E->W	Yes	Possible flow direction invertion during low demand periods
10	29-30	450	Asb.Cement	1505	1975	450	Gate	25463	50 40	Gate Angle	S N	S->N	No	Many branches and service connections to be closed
11	30-183	450	Asb.Cement	--	1975	450	Butterfly	25689	50 50	Gate-30 Ball	N S	N->S	No	Different materials; Service connections along the test section
12	30-116	600	Steel	--	1975	600	Butterfly	25468	50 50	Angle Angle	E W	W->E	No	Different diameters
13	49-50	750	Steel	1345	1972	750	Butterfly	20823	50	Angle	W	W->E	Yes	Chamber 13 has one tap; length is very short

(table continued on next page)

Table 3-5 - Continued

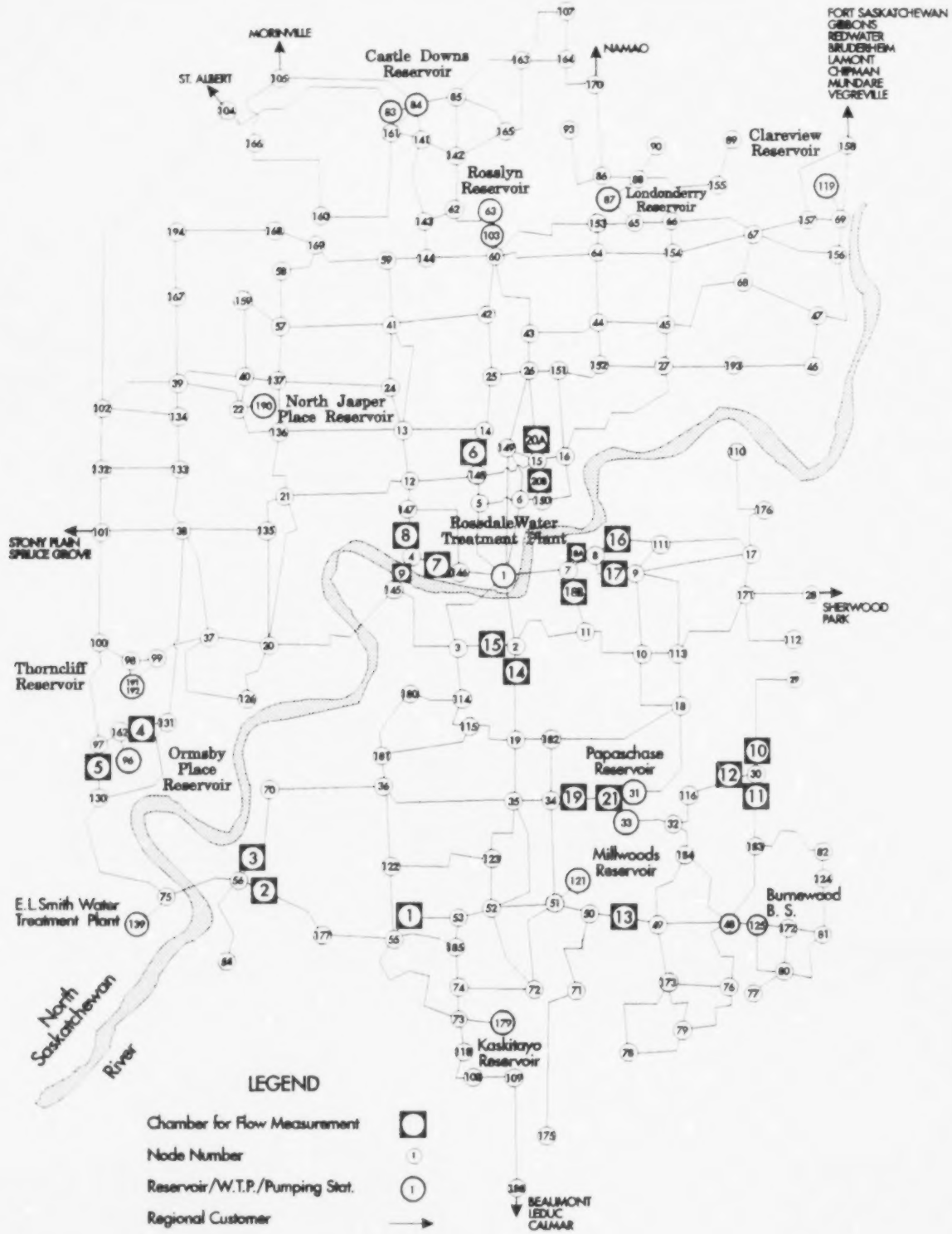
No. of the Chamber (1)	Model Node to Node(2)	Pipe Test Section				Control Valve			Air Vent Valves				Possible to Test	Observations
		Diameter (mm)	Material	Length (m)	Year	Diameter (mm)	Type	ID Number	Diameter (mm)	Type	Position to Control Valve	Flow Direction		
14	2-19	900	Pres.Concrete	2517	1967	900	Butterfly	20301	50	Gate	S	N->S	Yes	Good for C factor test; not possible to close branches for 24 hours to develop test
									50	Angle	N			
									50	Angle	S			
15	2-3	750	Pres.Concrete	1347	1968	750	Butterfly	20292	50	Gate-45	W	E->W	Yes	Need to close 2 important branches; Check valve in chamber, very short length
									50	Angle	E			
									50	Angle	W			
									750	Check	W			
16	8-111	750	Pres.Concrete	--	1971	750	Butterfly	3372	50	Gate	N	W->E	No	Different materials
									50	Angle	S			
17	8-9	500	Steel	--	1951	450	Butterfly	6769	50	Gate-45	E	W->E	No	Very short length of same diameter
									50	Angle	E			
									50	Angle	W			
18-A	7-8	750	Steel	950	1969	750	Butterfly	20254	50	Gate	N	S->N	No	Length is not adequate; need to close two important valves
									50	Angle	N			
									50	Angle	S			
18-B	7-11	750	Steel	1600	1960	750	Gate	13850	50	Gate-45	S	N->S	Yes	Check valve in chamber; length is short
									50	Angle	N			
19	33-34	450	Asb.Cement	--	1963	450	Butterfly	16045	50	Angle	E	W->E	No	Many services and branches to close
									50	Angle	W			
20-A	15-26	1050	Pres.Concrete	2047	1969	1050	Butterfly	21655	50	Gate	N	S->N	Yes	Good for C-factor test; Tap is very close to the butterfly valve
									50	Angle	N			
									50	Angle	S			
20-B	15-150	600/450	Pres.Concrete	--	1974	600	Butterfly	21656	50	Gate	S	N->S	No	Different diameters
21	33-34	600	Pres.Concrete	3556	1989	450	Butterfly	42600	50	Gate	W	W->E	Yes	Very good for C-factor test

(based on the City of Edmonton Public Works Cadastral Drawings)

(1) see Figure 3-2.

(2) see Figures 3-1 and 3-2.

Figure 3-2. Schematic of Chambers in Water Transmission Mains Already Prepared for Flow Measurements



3.2.2. Selected Lines

The City of Edmonton has 179 km of pipelines with diameter larger than or equal to 600 mm. Those pipelines correspond to 118.5 km of pressure concrete pipe, 53.3 km of steel pipes, 7.0 km of fiberglass reinforced pipe (FRP), and 0.2 km of asbestos-cement. However, just over 20 percent of those pipelines meet the selection criteria for head loss tests.

Analyzing the twenty-three chambers listed in Table 3-5, eleven met the selection criteria for C-factor measurement. Unfortunately, six chambers have pipe test sections that are too short to obtain a reasonable head loss. However, those six chambers may be used if flows producing an average velocity of approximately 1 to 1.25 m/s can be achieved. They were included in the list of pipelines suitable to be tested since it may be feasible to achieve the higher velocity.

Although not suitable for testing now, five other pipelines were included in the selection. They may be tested when the chambers for pressure and flow measurement are prepared for data collection.

Table 3-6 presents a summary of the pipe test lines. Test lines from 1 to 11 are those chambers that were previously prepared for flow measurement. Test lines from 12 to 16 were those which require valve installation or tapping for the development of the tests. Figure 3-3 shows the schematic location of the sixteen test lines chosen.

3.2.3. Field Tests in Water Transmission Mains

Field tests in water transmission mains were developed in two phases. Initially, flow tests were developed in the Londonderry pumping station. Finally, head loss tests were developed in four transmission mains for C-factor measurement.

The tests in the Londonderry pumping station were developed to verify the effects of valves in flow measurement, since flow in the field tests was measured in control valve chambers. The tests were developed in the chamber of the butterfly valve No.15989, shown schematically in Figure 3-4.

Four lines of the sixteen selected for field tests were tested. According to the Water Branch of the City of Edmonton Public Works, some of the sixteen lines could not meet the conditions required for the development of the tests at the period available for testing. Therefore, the longest lines were chosen first, leaving the others for a posterior opportunity. The lines tested were line I, II, IX, and XII summarized in Table 3-6 and located in Figure 3-3. Figures 3-5 to 3-8 present a schematic of each test line, respectively. Following each schematic, a data summary introduces the main characteristics of each line.

Table 3-6. Summary of Pipe Test Sections Selected for Head Loss Tests

Line Number (1)	Pipeline Characteristics				No. of Existing Flow Chamber	Predominant Flow Direction	Number of Branch Valves to Close
	Diameter (mm)	Material	Year	Length (m)			
I	1050	Steel	1975	3542	2	W to E	3
II	1050	Pressure Concrete	1969	2407	20-A	S to N	2
III	900	Pressure Concrete	1967	2517	14	N to S	2
IV	750	Steel	1960	2548	9	N to S & E to W	2
V	750	Steel	1960	1600	18-B	N to S & E to W	None
VI	900	Lined Steel	1958	1790	8	S to N	6
VII	1050	Pressure Concrete	1958	1325	7	E to W	None
VIII	750	Pressure Concrete	1968	1347	15	E toW	2
IX	600	Pressure Concrete	1989	3556	21	W to E & N to S	1
X	900	Pressure Concrete	1975	1505	1	W to E	2
XI	750	Welded Steel	1950	1700	6	S to N	8
XII	1050	Pressure Concrete	1980	4620	--	S to N & W to E	None
XIII	1050	Pressure Concrete	1987	3607	--	W to E & S to N	None
XIV	600	Steel	1950	1763	--	E to W	8
XV	600	Steel	1961	2208	--	S to N	2
XVI	750	Pressure Concrete	1989	2460	--	W to E & S to N	2

(1) see Figure 3-3

Figure 3-3. Summary of Pipe Test Sections Selected for Head Loss Tests

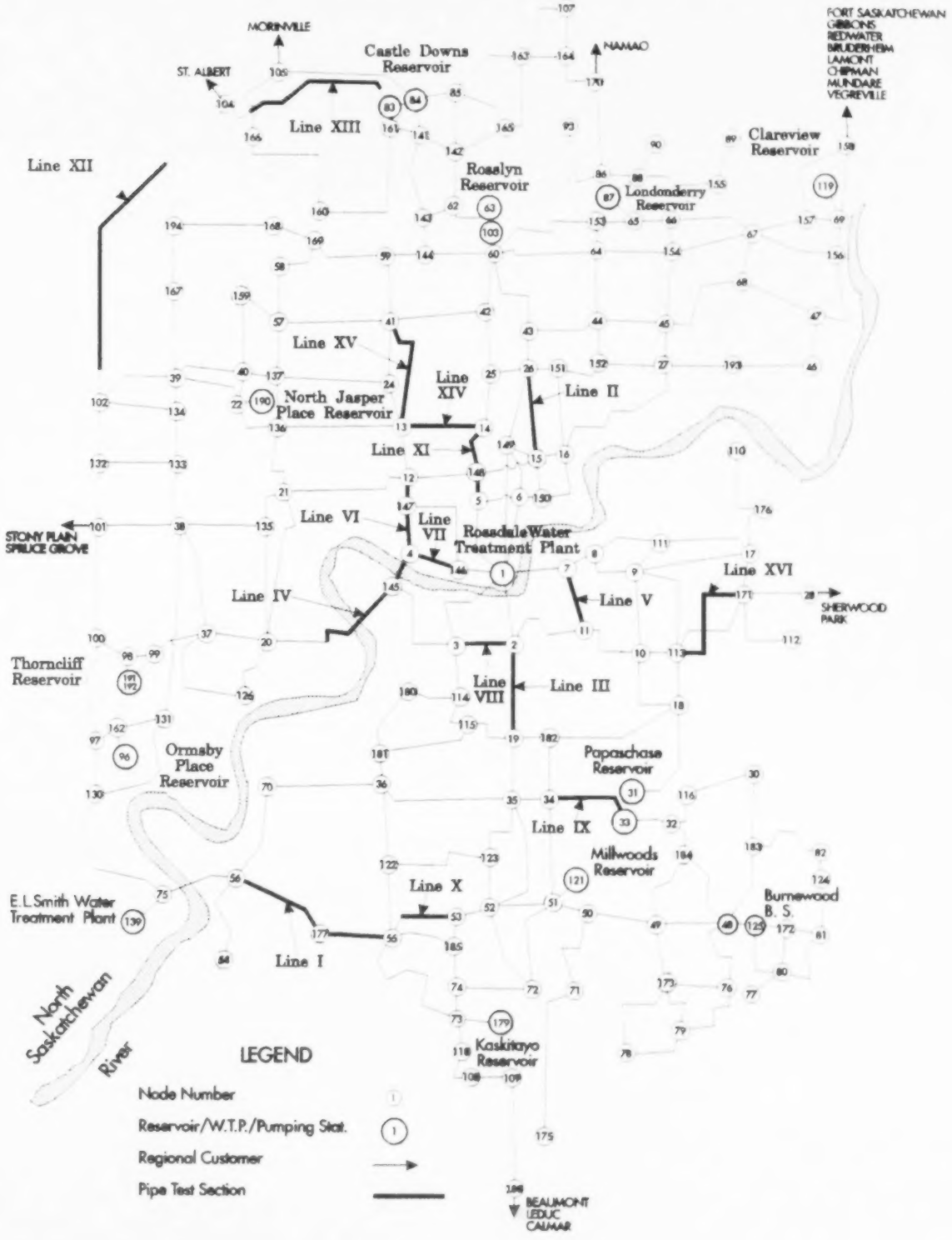


Figure 3-4. Schematic of the Londonderry Pumping Station

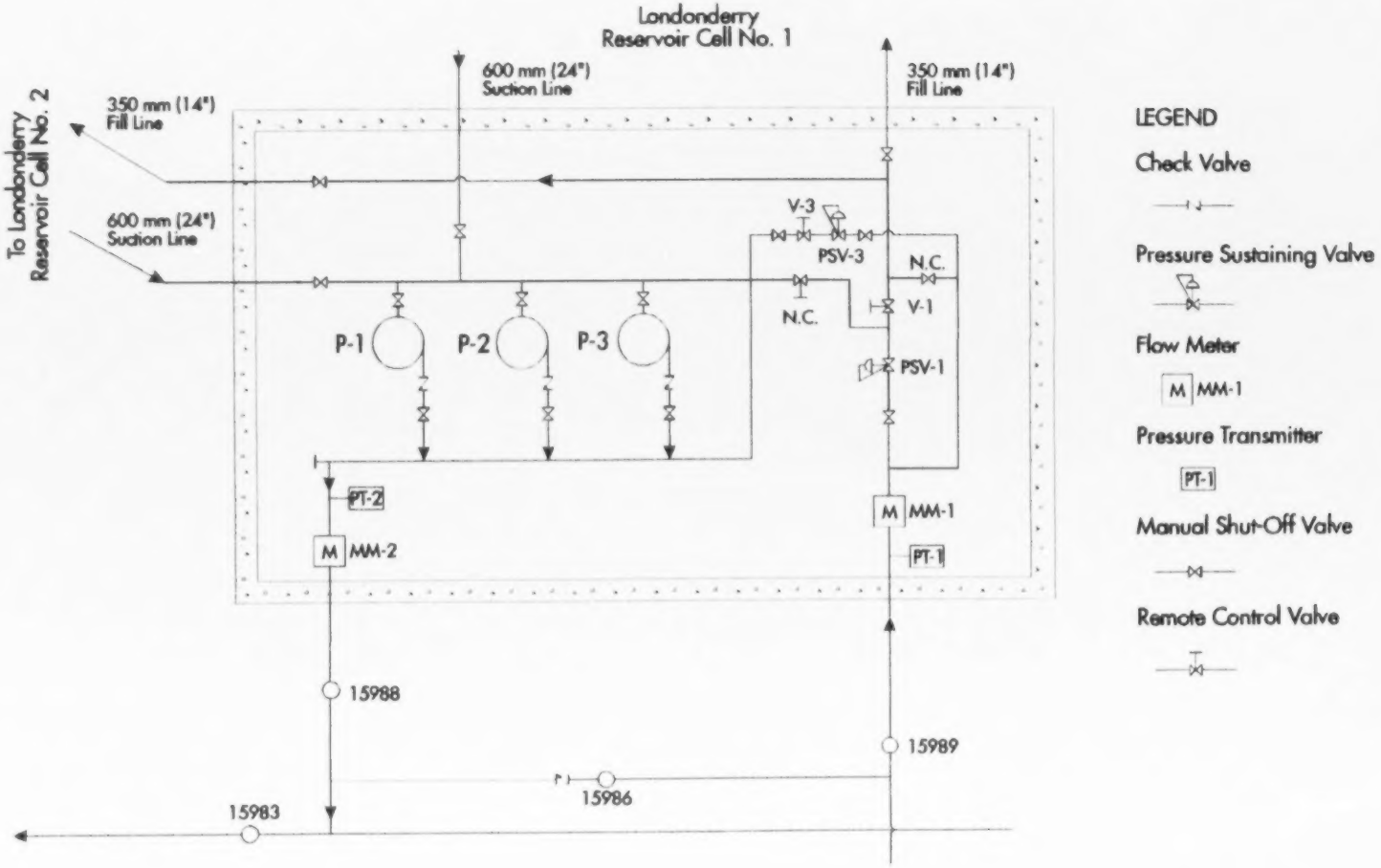
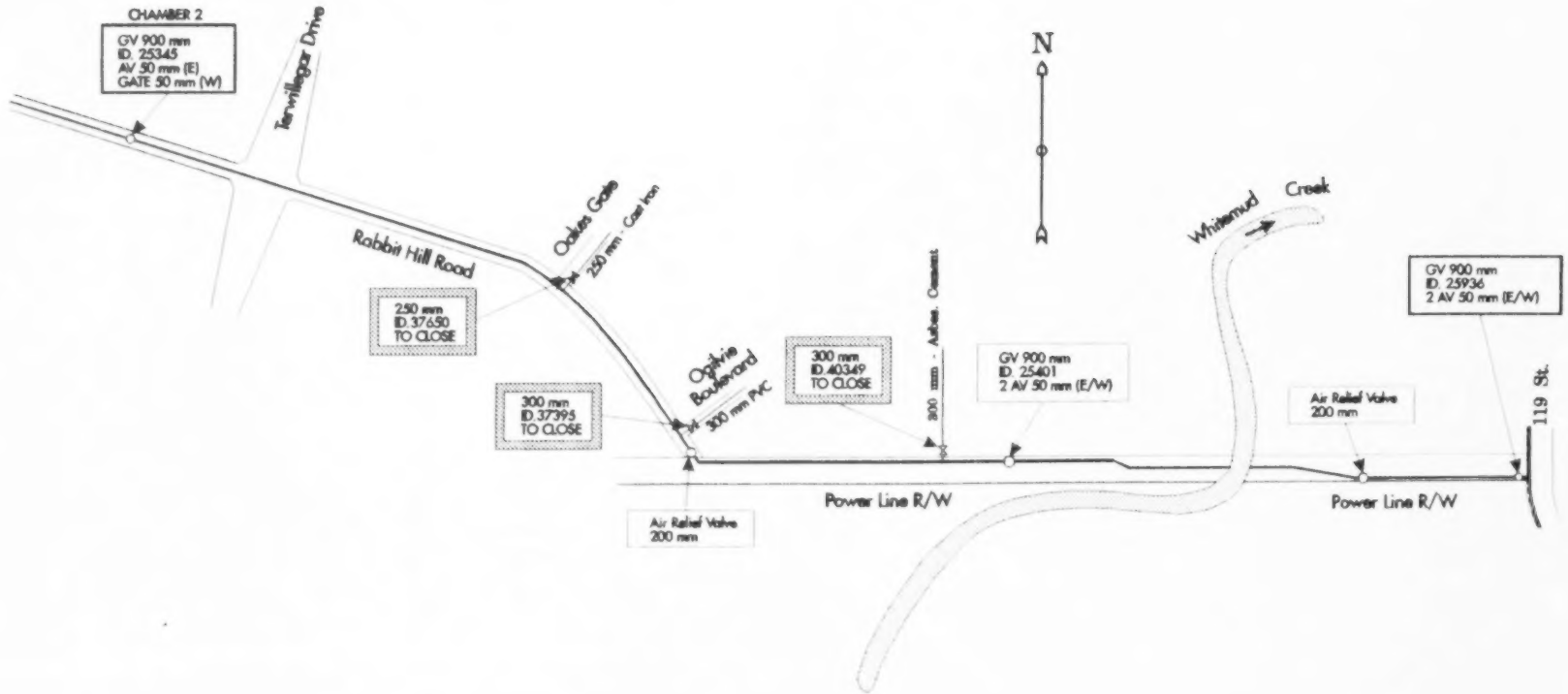


Figure 3-5. Schematic of Line I

Steel - 1050 mm



Data Summary for Line I

Diameter	1050 mm (42")
Material	Steel
Year of Construction.	1975
Length of Test Section	3542 m
Direction of Flow	W to E
Number of the Flow Measurement Chamber	2

Chamber for Flow and Pressure Measurement

Control Valve

Diameter	900 mm
Type	Gate
ID Number	25345

Auxiliary Valves

Diameter	50 mm	50 mm
Type	Gate-45º	Angle
Position to Control Valve	W	E

Chamber for Pressure Measurement

Control Valve

Diameter	900 mm
Type	Gate
ID Number	25936

Auxiliary Valves

Diameter	50 mm	50 mm
Type	Angle	Angle
Position to Control Valve	W	E

Branch Valves to Close

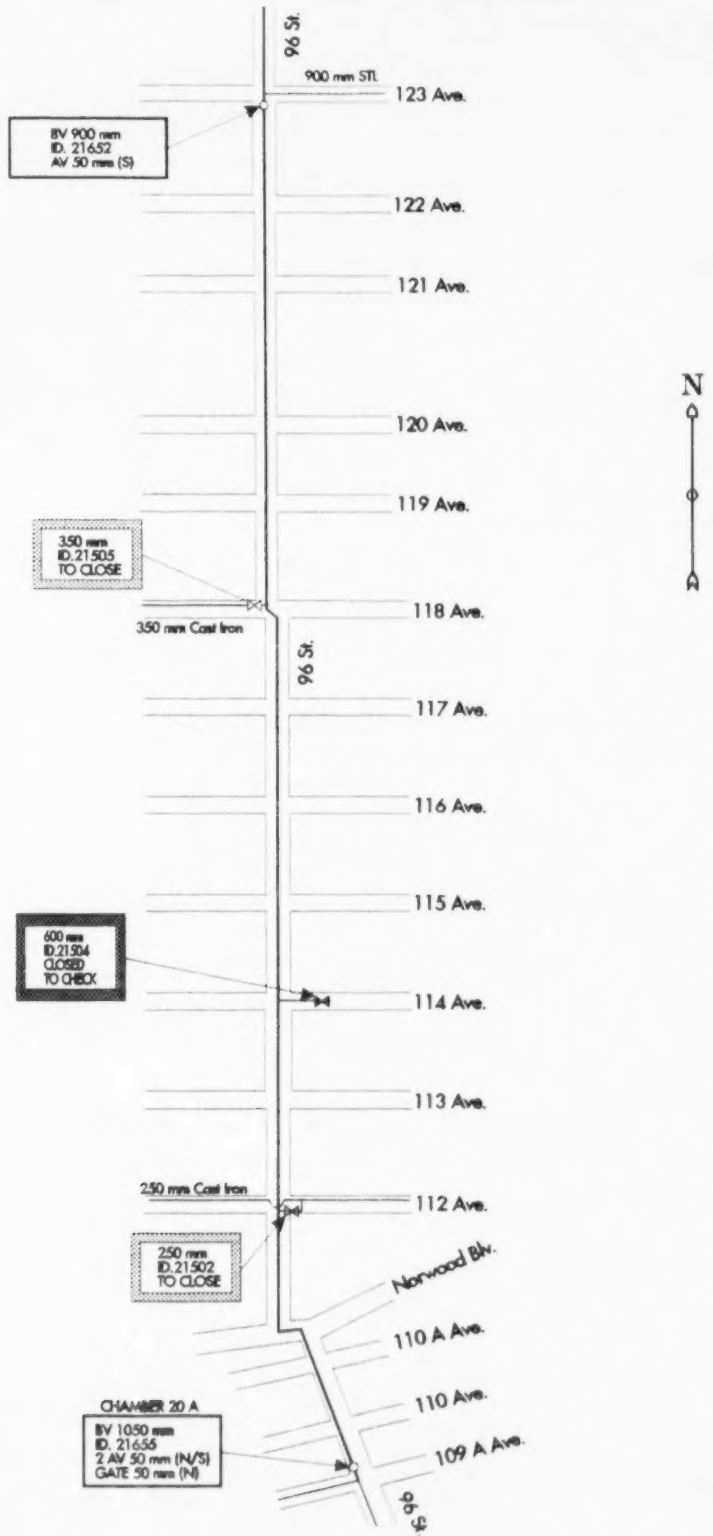
ID Number	37650	37395	40349
Diameter	250 mm	300 mm	300 mm

Observations

Very good pipe section for C-factor measurement, good length, good chambers for installation of measuring equipment, good flow, and just three branches to close.

IMPORTANT: According to Transmission Main Engineering Department is not possible to close branches for 24 hours, therefore, tests must be developed from 9:00 AM to 3:00 PM.

Figure 3-6. Schematic of Line II
(Pressure Concrete Pipe - 1050 mm)



Data Summary for Line II

Diameter	1050 mm
Material	Pressure Concrete Pipe
Year of Construction	1969
Length of Test Section	2407 m
Direction of Flow	S to N
Number of the Flow Measurement Chamber	20-A

Chamber for Flow and Pressure Measurement

	Control Valve
Diameter	1050 mm
Type	Butterfly
ID Number	21655

Auxiliary Valves

Diameter	50 mm	50 mm	50 mm
Type	Gate-45°	Angle	Angle
Position to Control Valve	N	N	S

Chamber for Pressure Measurement

	Control Valve
Diameter	900 mm
Type	Butterfly
ID Number	21652

Auxiliary Valve

Diameter	50 mm
Type	Angle
Position to Control Valve	S

Branch Valves to Close

ID Number	21502	21505	21504
Diameter	250 mm	350 mm	600 mm *

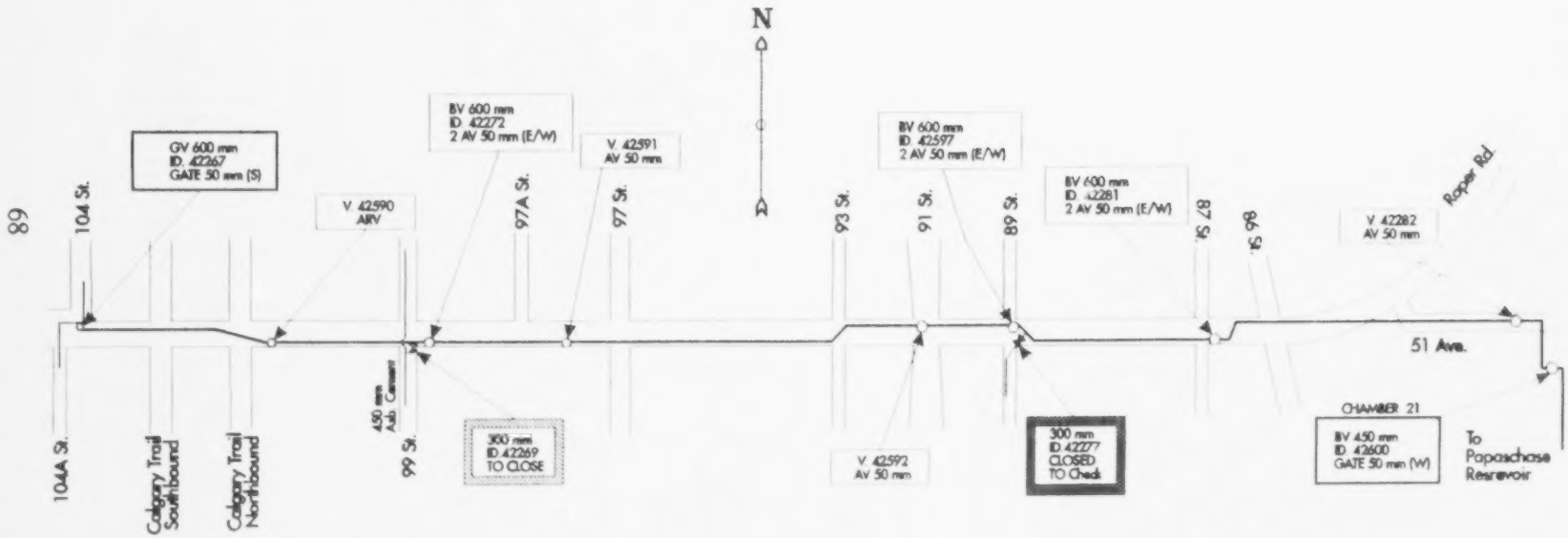
* Valve signed in the Water Branch computer as normally in closed position.

Observations

Good length, good flow, good chambers for installation of measuring equipment, possible to test for 24 hours, and just two branches to close.

IMPORTANT: One problematic point is that the tap for the insertion of the flowmeter in Chamber 2 (control valve 21655) is too close to the butterfly valve, not allowing the measurement of the velocity profile, consequently, not possible to position the probe in the average flow position. If tap repositioned, the line is good for test.

Figure 3-7. Schematic of Line IX
Pressure Concrete Pipe - 600 mm



Data Summary for Line IX

Diameter	600 mm
Material	Pressure Concrete Pipe
Year of Construction	1989
Length of Test Section	3556 m
Direction of Flow	W to E and N to S
Number of the Flow Measurement Chamber	21

Chamber of Flow and Pressure Measurement

Control Valve

Diameter	450 mm
Type	Butterfly
ID Number	42600

Auxiliary Valve

Diameter	50 mm
Type	Gate-Top
Position to Control Valve	W

Chamber for Pressure Measurement

Control Valve

Diameter	600 mm
Type	Gate
ID Number	42267

Auxiliary Valve

Diameter	50 mm
Type	Gate-Top
Position to Control Valve	S

Branch Valves to Close

ID Number	42269	42277*
Diameter	300 mm	300 mm

* Valve signed in the Water Branch computer as normally in closed position.

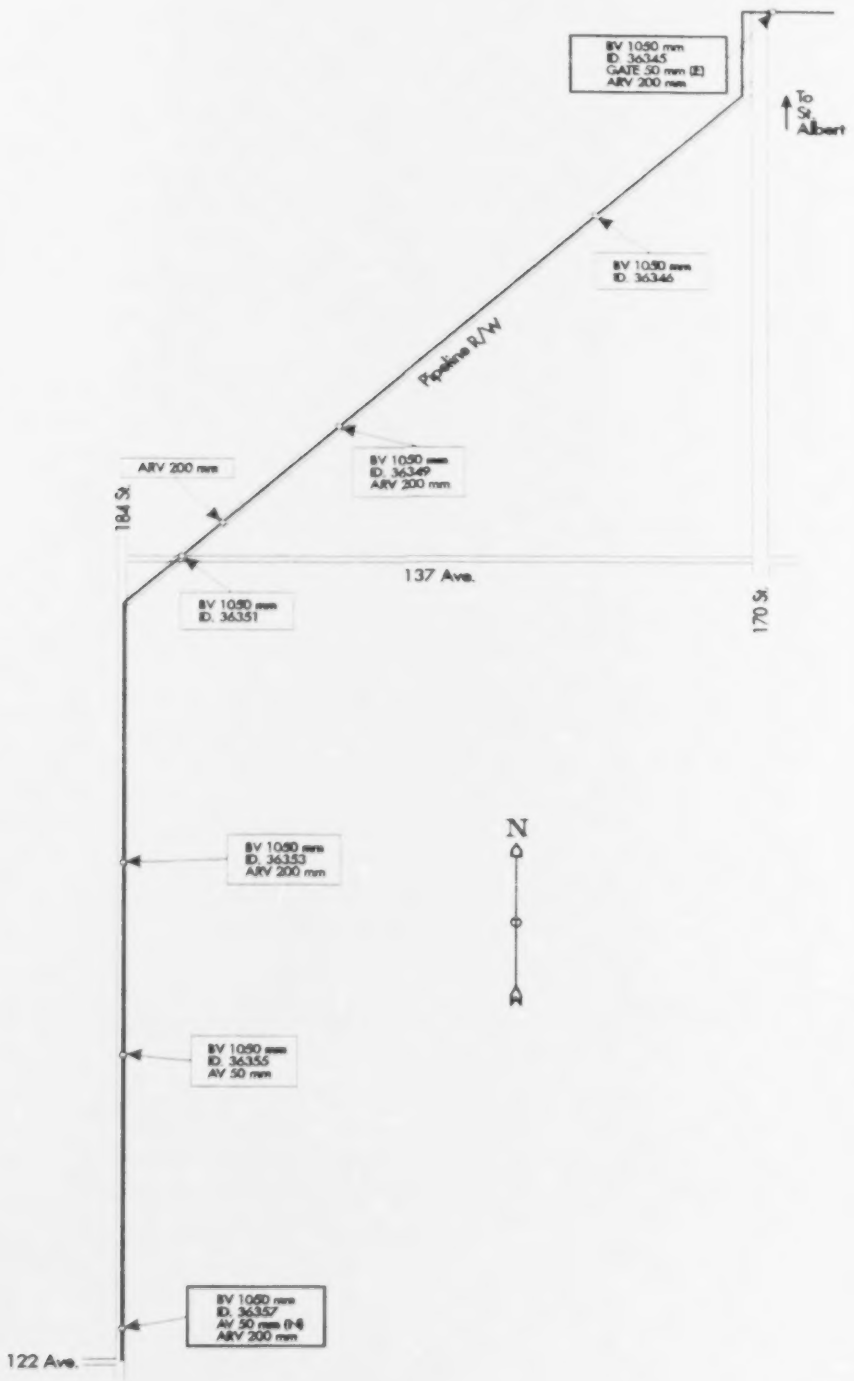
Observations

Very good pipe section for C-factor measurement, good length, good chambers for installation of measuring equipment, possible to measure for 24 hours, good flow, and just one branch to close. Another positive factor is that there is a flowmeter installed in the Papaschase reservoirs, allowing to compare the obtained flows.

IMPORTANT: In both chambers there is just one gate valve air vent, but it is possible to use a tee to measure flow and pressure only in Chamber 21 (Papaschase), in the other chamber there is no room to insert the flowmeter.

Figure 3-8. Schematic of Line XII

Pressure Concrete Pipe - 1050 mm



Data Summary for Line XII

Diameter	1050 mm
Material	Pressure Concrete Pipe
Year of Construction	1980
Length of Test Section	4620 m
Direction of Flow	S to N and W to E
Number of the Flow Measurement Chamber	Not numbered

Chamber for Flow and Pressure Measurement

	Control Valve
Diameter	1050 mm
Type	Butterfly
ID Number	36345

Auxiliary Valves

Diameter	50 mm	200 mm
Type	Angle (changed for a Gate-Top)	ARV
Position to Control Valve	E	W

Chamber for Pressure Measurement

	Control Valve
Diameter	1050 mm
Type	Butterfly
ID Number	36357

Auxiliary Valves

Diameter	50 mm	200 mm
Type	Angle	ARV
Position to Control Valve	N	S

Branch Valves to Close

No branches to close.

Observations

Very good pipe section for C-factor measurement, good length, very good chambers for installation of measuring equipment, good flow, possible to test for 24 hours, and no branches to close.

IMPORTANT: In the chamber of the valve 36345, there is just one gate valve air vent. Therefore, it is necessary to use a tee to measure flow and pressure.

3.3. INSTRUMENTATION

3.3.1. General

a. Pressure

The devices available for pressure measurement in the head loss tests consisted of two pressure transducers of the type PTX 160/D manufactured by Druck Ltd. This type of transducer is useful for insertion into small diameter holes due to the small diameter of the body (17.5 mm). It may be used for pressure measurements in any liquid compatible with quartz (transduction cell) and titanium (body) (Druck Limited, 1984).

For head loss tests in large diameter mains, an adaptor of 50 mm is required. It consists of a piece of 50 mm pipe capped at one end which has the insertion hole for the pressure transducer, the holding connection, and a small valve to remove air from the adaptor when installed in a pipeline.

The calibration of each pressure transducer was done in the Geotechnical Laboratory of the Civil Engineering Department at the University of Alberta. The equipment used in the calibration is normally used in the calibration of transducers for geotechnical work. Figure 3-9 shows the calibration curve for transducer 1 and Figure 3-10 for transducer 2.

The calibration curve obtained for transducer 1 is:

$$P' = 17.536\ V_{VOLT} - 18.247 \qquad (3\text{-}1)$$

where:

P' = pressure head (m H_2O)

V_{VOLT} = voltage (Volts)

The calibration curve obtained for transducer 2 is:

$$P' = 17.610\ V_{VOLT} - 18.193 \qquad (3\text{-}2)$$

b. Flow

The device available for flow measurement in pipelines was an electromagnetic meter of the type MSR Magmeter. This type of meter is classified as an insertion sensor style meter, especially designed for insertion into a pipe while this pipe is in use and under pressure (MSR Magmeter, 1984).

This type of meter does not measure flow, but velocity in a determined point in the pipe cross section. Therefore, it is necessary to measure the velocity profile and integrate this profile to obtain flow (Walski, 1984). However, if the position of the average velocity is determined in the pipe cross section, the flow may be obtained from a single reading.

Figure 3-9. Calibration Curve of Transducer 1

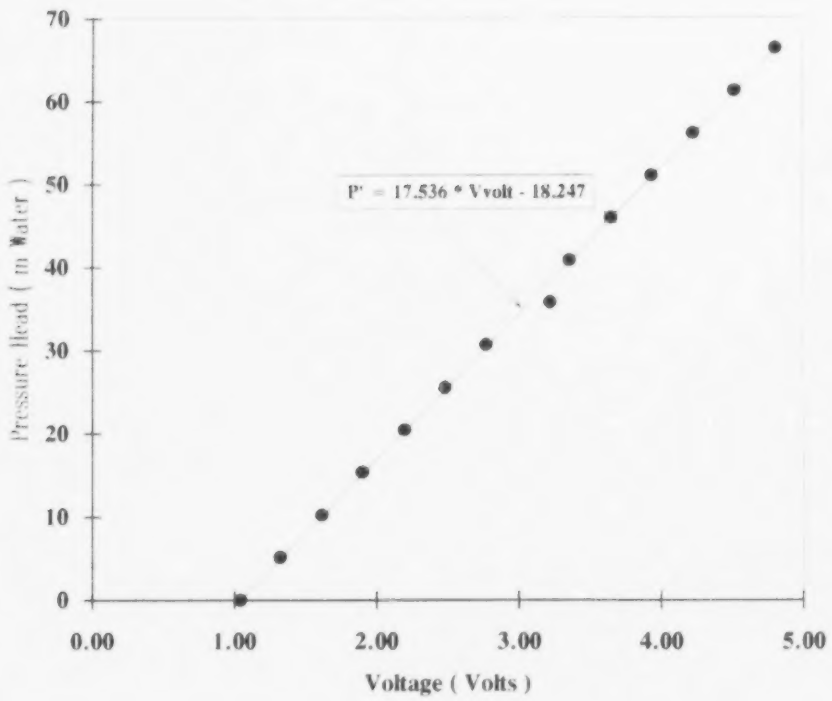
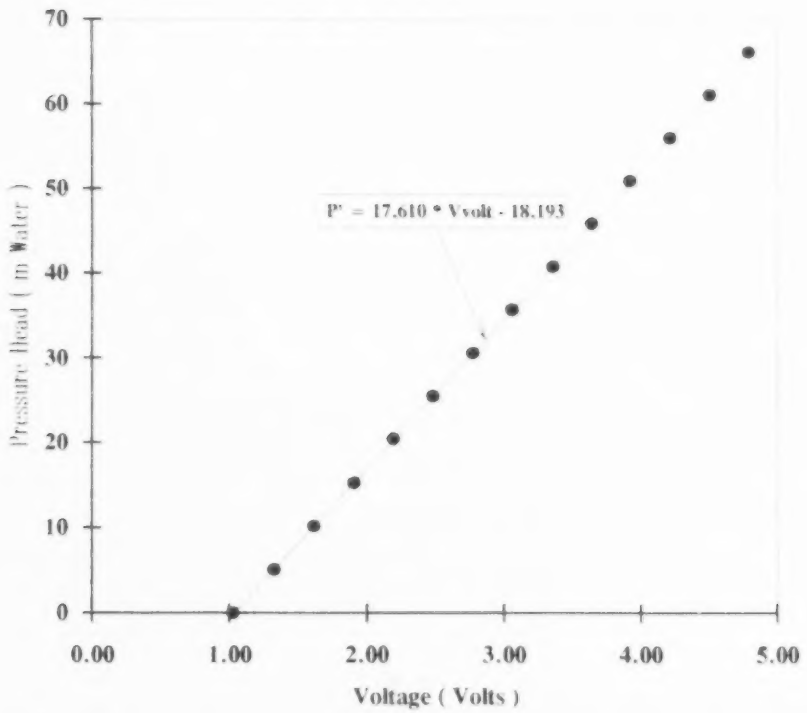


Figure 3-10. Calibration Curve of Transducer 2



Rajaratnan and Lovell (1985) and Flint-Petersen and Rajaratnam (1987) studied the position of average velocity in a pipe using MSR Magmeters. This position was found to be 22 percent of the pipe radius from the internal pipe wall, or 78 percent of the radius from the axis of the pipe. This position of the sensor refers to an ideal velocity profile. However, this position differs from situation to situation in field tests. Therefore, the position of the average velocity in the pipe has to be defined by measuring the velocity profile in field tests (MSR Magmeter, 1984).

The MSR Magmeter was calibrated by Sheldon Lovell in the Hydraulics Laboratory of the Civil Engineering Department at the University of Alberta. One calibration curve was developed for the range 0 to 1 m/s and another for the range 0 to 2 m/s. Figure 3-11 shows the calibration curve for the 0 to 1 m/s velocity range and Figure 3-12 for the 0 to 2 m/s.

The calibration curve obtained for the velocity range 0 to 1 m/s is:

$$v = 0.0060\ V_{mv} - 0.19 \qquad (3\text{-}3)$$

where:

v = average velocity (m/s)
V_{mv} = voltage (millivolts)

The calibration curve obtained for the velocity range 0 to 2 m/s is:

$$v = 0.0114\ V_{mv} - 0.26 \qquad (3\text{-}4)$$

c. Data Acquisition

Two Microscout® (Terrascience Systems Ltd.) data loggers were used to record flow and pressure readings. Data was down loaded to a PC-compatible laptop computer.

3.3.2. Procedures for Field Setup

a. Pressure

The setup of the pressure transducers is very simple. It consists of installing the adaptor into the valve, then inserting and fixing the transducer into the adaptor. Figure 3-13 shows the installation of a transducer in an isolated tap.

The following procedures were used for the Druck transducer setup in the field for isolated taps (one tap for pressure and one tap for flow) or a single tap using a tee (pressure and flow measured in the same tap).

1. Installation of the Adaptor

   - o Install the adaptor into the gate or angle valve before installing the transducer.

Figure 3-11. Calibration Curve of MSR Magmeter for the Velocity Range of 0 to 1 m/s

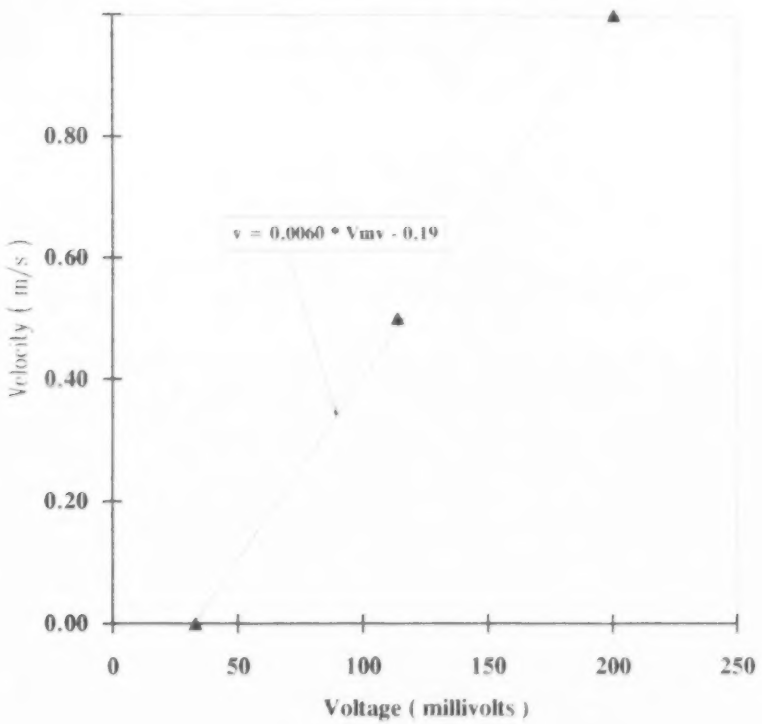


Figure 3-12. Calibration Curve of MSR Magmeter for the Velocity Range of 0 to 2 m/s

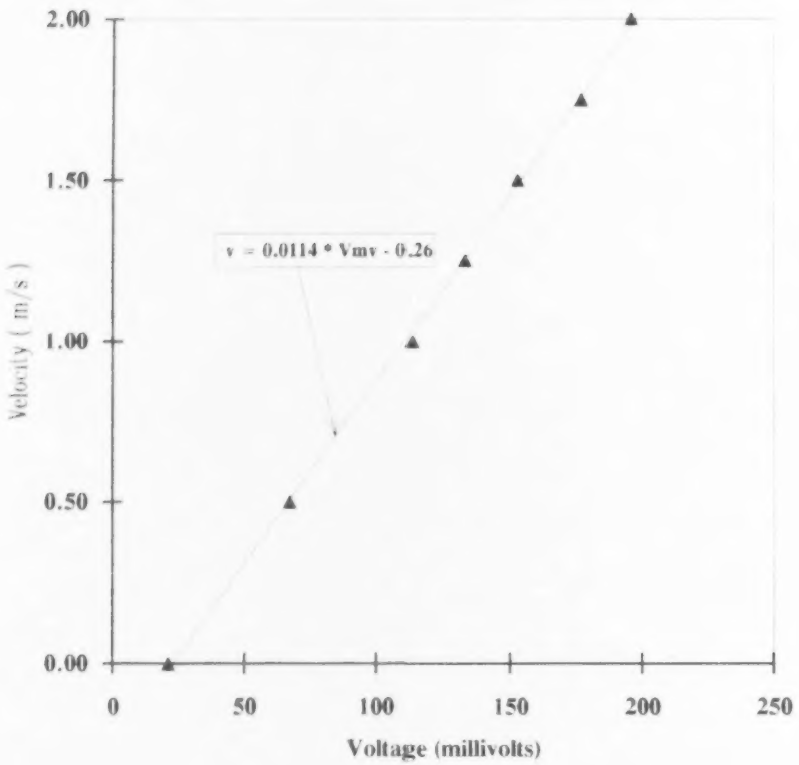
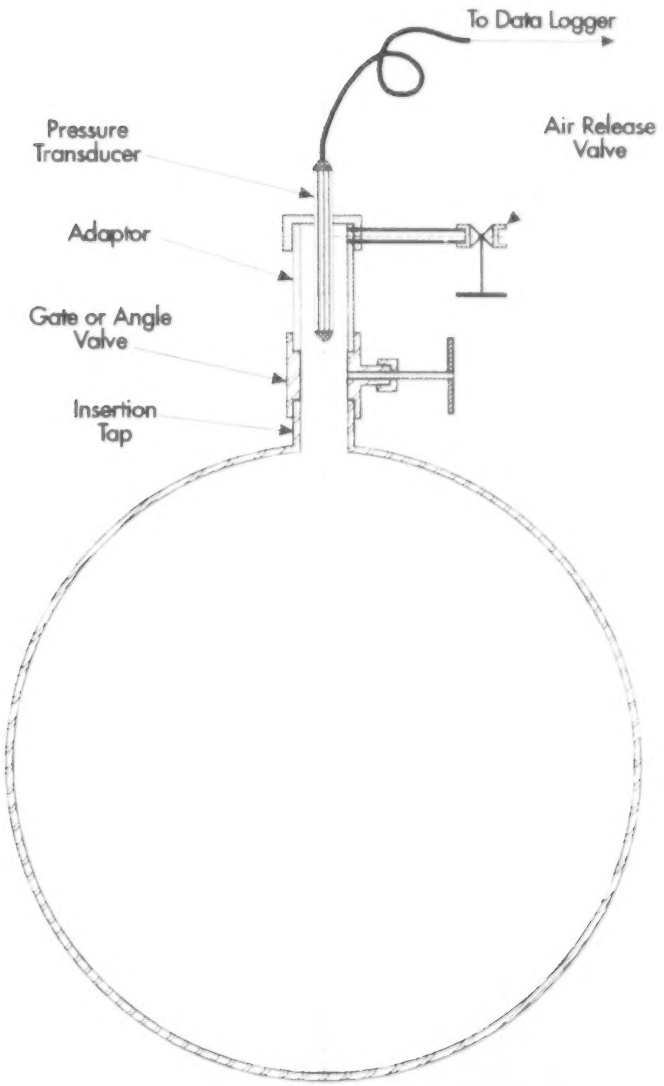


Figure 3-13. Schematic of a Pressure Transducer Installed in an Isolated Tap



- o The air relief valve may be positioned facing the upside, for a better air removal.

2. Installation of the Transducer

   - o Insert the transducer into the adaptor and fix it with the holding connector, sealing with an O-ring.

   - o After installing the transducer into the gate valve (or angle valve) and before proceeding, open slowly the valve and check for leaks.

   - o Remove air from the adaptor, opening the air relief valve and allowing water to flow for a few seconds.

   - o Connect the wire of the transducer to the data logger.

b. Flow

There were two possible situations for the installation of the meter. The first was in a chamber were flow and pressure instruments were installed in different taps. The second was in a chamber were a single tap was available for the installation of both instruments, requiring the use of a tee. Figure 3-14 shows a schematic of both situations.

The following procedures were used for the MSR Magmeter setup in the field.

1. Safety Procedures

   - o Never stay in front of the meter extension shaft. Pipes have high pressure inside that pushes the shaft out.

   - o Keep the extensions and the insertion mechanism (jack) free from oil, grease, or any oily substance.

2. Installation of the MSR Magmeter (different taps - Figure 3-14.a)

   - o Positioning the electrodes in relation to flow direction:

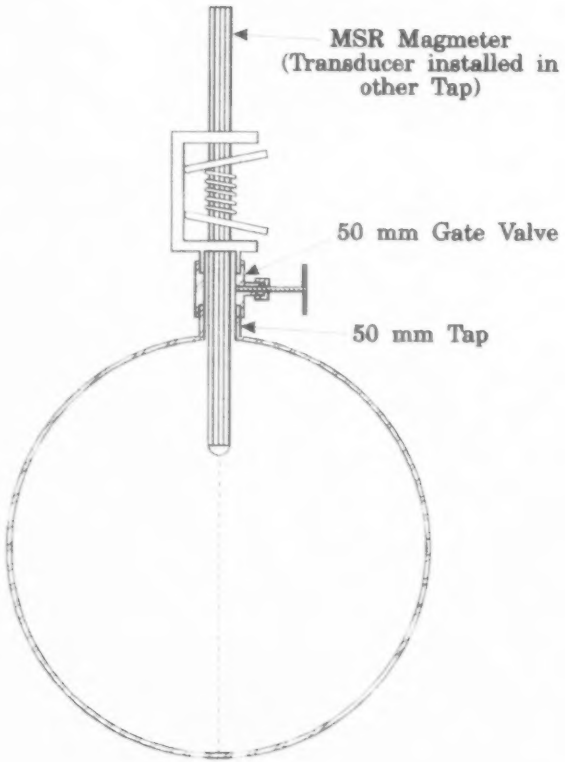
     - ·· the position of the electrodes in the probe has to be perpendicular to the flow direction in the pipe (Figure 3-15);

     - ·· testing position of the electrodes (the MSR Magmeter used in the tests reads flow in just one direction):

       - position electrodes perpendicular to the flow direction;

Figure 3-14. Schematic of the Installation of MSR Magmeter in Pipes

a. One Tap for Flow and
One Tap for Pressure Measurement



b. One Single Tap for Flow and
Pressure Measurement (Using a Tee)

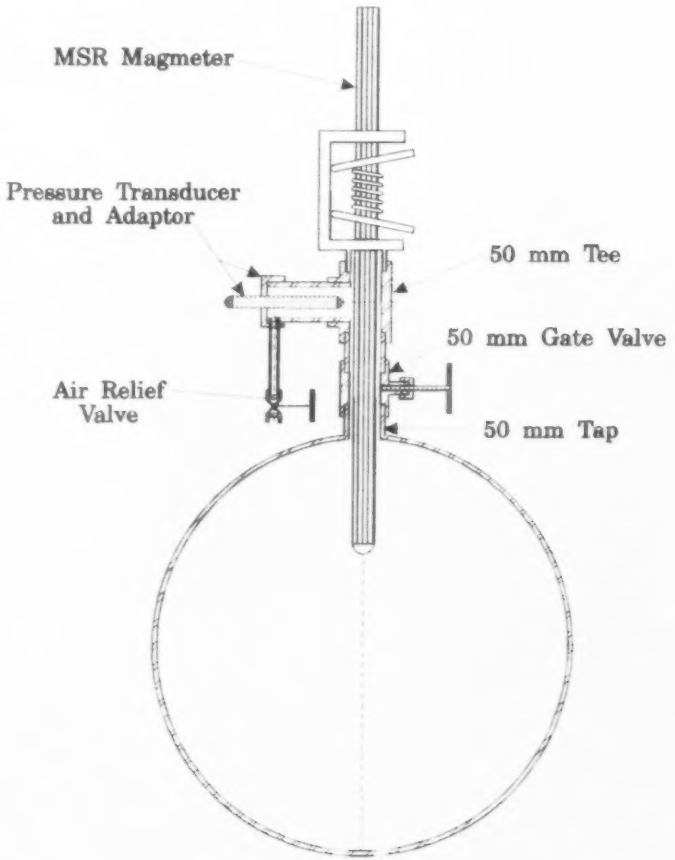
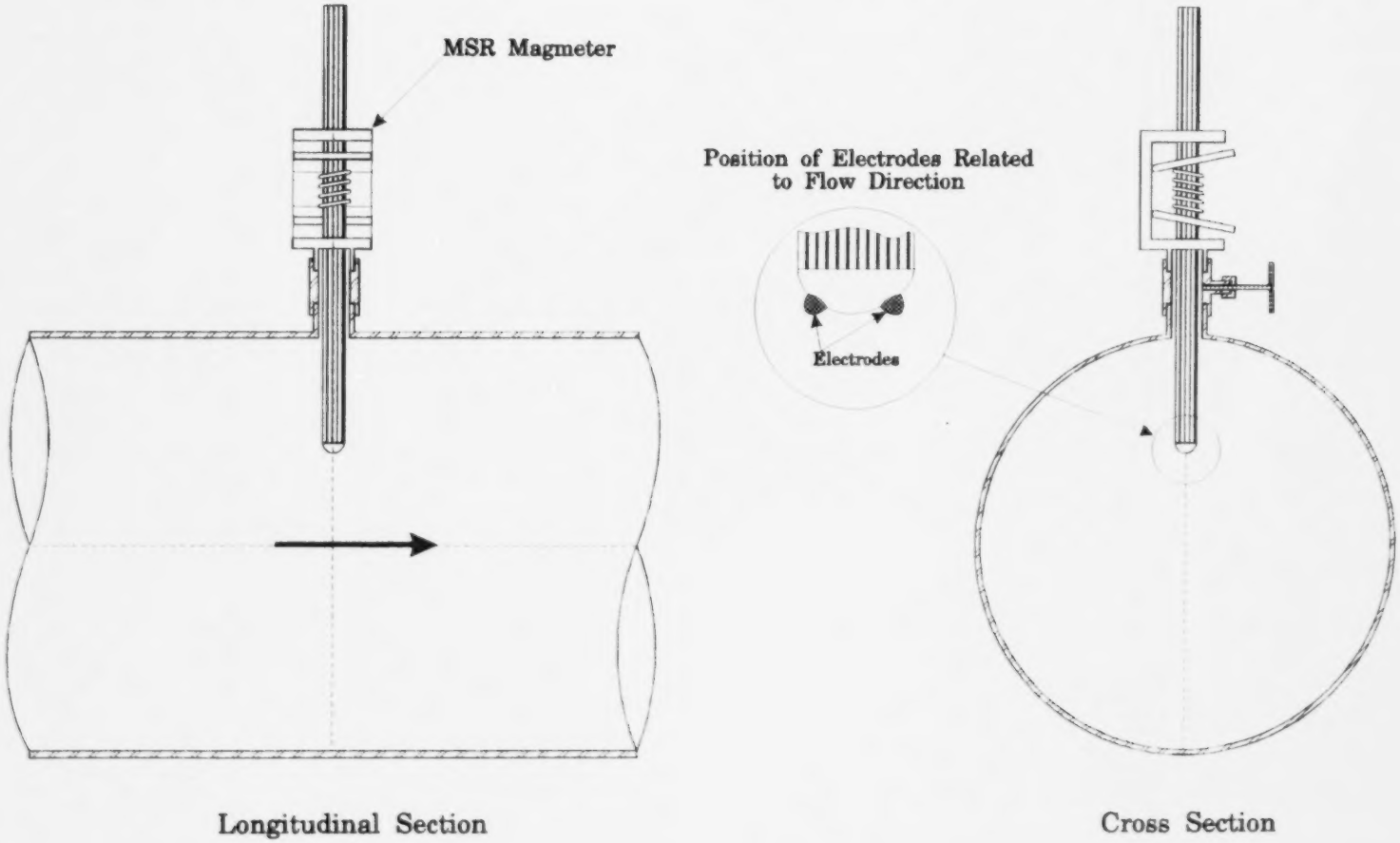


Figure 3-15. Position of the Electrodes in Relation to the Flow in the Pipe



- if the reading obtained in a multimeter with some water flowing into the pipe is larger than 30 millivolts, proceed;
- if reading obtained in a multimeter is smaller than 30 millivolts, check the electric connections of the coil and electrode wires, and check the battery power;
- if equipment is working perfectly, but reading is still smaller than 30 millivolts, rotate sensor by 180°.

o Calculation of position of the sensor probe into the pipe:

·· position of the sensor probe into pipe (Figure 3-16):

- measure distance top of the valve to top of pipe (X_1);
- measure length of thread screwed inside valve body (X_2);
- calculate distance probe retracted to top of pipe ($X_3 = X_1 - X_2$);
- measure external pipe radius (r);
- using the velocity profile, calculate the length up to the point where the average velocity has to be measured (X_4);
- calculate total length of insertion of the probe ($X_T = X_3 + X_4$);
- mark X_T on the extension shaft with the extension shaft totally raised.

o Once the Magmeter is correctly installed and tightened in the tap, check for leaks opening slowly the valve.

o Insert the sensor probe into the pipe by lowering the extension shaft up to the X_T mark.

o Connect electrode and coil cables to transmitter.

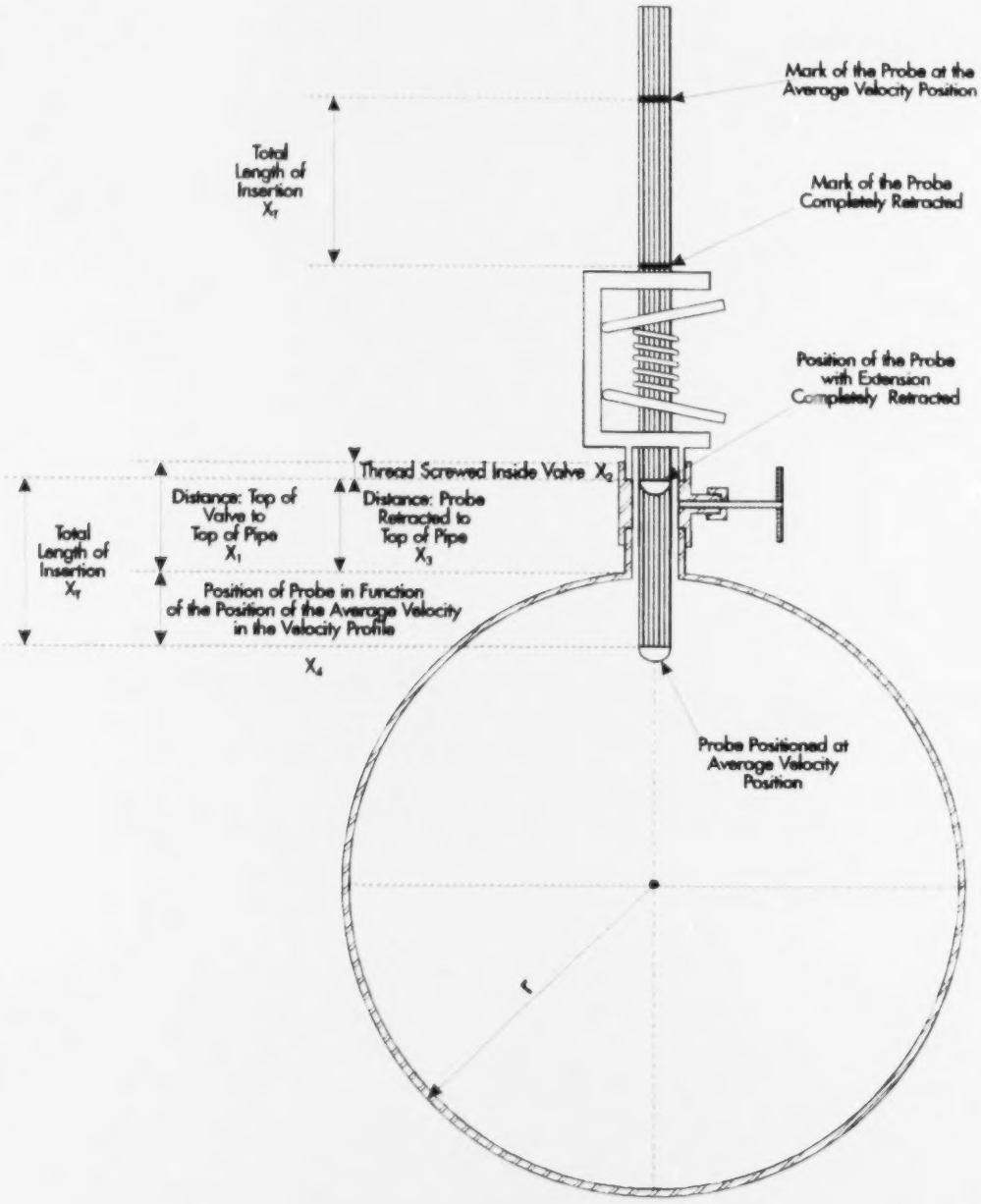
o Connect transmitter to the data logger.

3. Installation of the MSR Magmeter (single tap - Figure 3-14.b)

o Install the tee into the tap.

o Install the transducer adaptor and the transducer at the tee branch.

Figure 3-16. Positioning the Probe Inside the Pipe


Mark of the Probe at the Average Velocity Position
Total Length of Insertion X_T
Mark of the Probe Completely Retracted
Position of the Probe with Extension Completely Retracted
Thread Screwed Inside Valve X_2
Distance: Top of Valve to Top of Pipe X_1
Distance: Probe Retracted to Top of Pipe X_3
Total Length of Insertion X_T
Position of Probe in Function of the Position of the Average Velocity in the Velocity Profile
X_4
Probe Positioned at Average Velocity Position
r

- o Follow the same procedures as those described for the Installation of the MSR Magmeter for different taps.

4. Connecting the Meter to the Transmitter

   - o Connect wires to the transmitter, check that they are firmly attached to the connectors.

c. Data Acquisition

Data acquisition setup consisted of four steps. The first step was to program the logger for collecting data from the installed equipment. The second step was data acquisition during the field test. The third step was data retrieval from the logger. Finally, the last step consisted in preparing retrieved data for manipulation and analysis.

The Microscout® was programmed using the pocket terminal supplied with the logger and a laptop computer. However, the pocket terminal was easier to operate inside the valve chambers. Programming was developed using a resident program that comes with the logger.

Data retrieval was developed in two ways. Initially, data was retrieved in the field, using the laptop computer. Latter, this procedure was substituted by taking the logger to the office, then retrieving the stored data. The second procedure was safer and easier.

3.2. TEST PROCEDURES

a. Safety procedures for chamber entry

1. Follow procedures of the Code of Practice for Confined Space Entry developed by the Water Network Maintenance Department of the City of Edmonton Public Works (City of Edmonton, 1990a; 1990b) and here summarized:

   - o traffic control;
   - o use of safety equipment;
   - o entry procedures;
   - o emergency rescue procedures;
   - o lowering work equipment procedures.

b. Chamber for pressure measurement

1. Install adaptor and pressure transducer into a gate or angle valve air vent, following procedures described in section 3.3.2.a.

2. Connect transducer to Microscout® and follow the manufacturer procedures for data acquisition programing (Terrascience Systems Ltd., 1984).

3. Start Microscout® data acquisition.

c. Chamber for pressure and flow measurement

1. Install adaptor and pressure transducer and MSR Magmeter into air vents, following procedures described in sections 3.3.2.a and 3.3.2.b, respectively.

2. Connect transducer and MSR Magmeter to Microscout®.

3. Measure the velocity profile of the pipe section:

   - o read velocity every 25.4 mm (1 in) by lowering the MSR Magmeter graduated shaft;
   - o plot the velocity profile using millimetric paper;
   - o use equation 2-2 (section 2.1.2.) for calculation of the average velocity;
   - o define in the velocity profile plot the position to measure the average velocity in the pipe cross section;
   - o calculate the position of the probe into the pipe as described in section 3.3.2.b.

4. Position the sensor probe into the pipe and hold the probe in a fixed position using the holding lever in the MSR Magmeter.

5. Follow the manufacturer procedures for data acquisition programing (Terrascience Systems Ltd., 1984).

6. Start Microscout® data acquisition.

d. Data retrieval

1. After measuring data for the desired test period, retrieve the data using one of the following procedures:

   - o connect the laptop computer to the Microscout® and retrieve data using a communication program, or;

- o disconnect the measurement instruments from the Microscout® and retrieve data in the office.

e. Instruments removal

1. Instruments can be removed only after the data recovery.
2. Close the air vent valve before initiating any instrument removal.
3. Disconnect all wires before initiating any instrument removal.
4. Disassemble the MSR Magmeter and adaptor and pressure transducer.
5. Clean and dry all instruments and equipment used in the tests before storing.

f. Data manipulation

1. Use software package furnished by Microscout® manufacturer for transforming data into decimal values (Terrascience Systems Ltd., 1984).
2. Use calibration curves for transformation of data from voltage signal to pressure head and water velocity values.
3. Analyze data for C-factor calculation:
   - o separate data for periods of constant flow rates;
   - o calculate pressure head and flow average for each period of constant flow;
   - o plot the residuals and analyze the plots to verify data consistency for each period (Box et al, 1978; Draper and Smith, 1981);
   - o discard inconsistent data and use in calculation only data considered as consistent.
4. Calculate C-factor:
   - o calculate head loss for periods of constant flow rates;
   - o use average flow rate for each period;
   - o use equation 2-21 (section 2.1.5.) for C-factor calculation.

4. RESULTS AND DISCUSSION

4.1. LONDONDERRY RESERVOIR TEST

Flow tests at the Londonderry reservoir were done on September 17, 1991 using the fill line. The velocity profile was measured for two different flow rates: 9.73 ML/d and 20 ML/d. Those flow rates were monitored by a permanent installed magnetic flowmeter existing in the pumping station (MM-1 in Figure 3-4). This flowmeter is periodically calibrated by the Water Branch of the City of Edmonton that also furnished the average flow monitored during each test.

The first test was done between 12:50 AM and 13:10 AM for an average flow rate of 9.73 ML/d. The MSR Magmeter was used with a velocity range of 0 to 1 m/s. The results of the velocity profile are presented in Table 4-1, and the velocity profile is plotted in Figure 4-1.

The second test was developed between 14:25 AM and 14:50 AM for an average flow rate of 20.0 ML/d. The MSR Magmeter was used with the same velocity range as the first test. The results of the velocity profile are presented in Table 4-2, and the velocity profile is plotted in Figure 4-1.

Flow rates measured with the MSR Magmeter and with the flowmeter in the pumping station were very close. The difference for the 9.73 ML/d flow rate was - 1.40 percent, and for the 20 ML/d flow rate was + 4.74 percent. Hinds (1946) suggested an acceptable error of 10 percent. Consequently, both values were considered good for the development of field tests.

Figure 4-1 shows that the position of the average velocity is different from the 22 percent reported for ideal turbulent flow (Flint-Petersen and Rajaratnam, 1987). Both velocity profiles showed a wide area with a velocity close to the average velocity. This area ranged from 25 percent of the pipe radius measured from the pipe wall up to the center of the pipe.

It was also possible to notice the effect of the butterfly valve disc on the velocity profile. Although the valve was considered as completely open, it is possible to verify in Figure 4-1 that flow was higher at one half of the pipe than in the other.

The following conclusions were drawn from the tests developed:

1. butterfly valves close to the point of flow measurement considerably affected the velocity profile;

2. the point to measure the average velocity in the velocity profile may have been different from 22 percent of the pipe radius measured from the internal pipe wall;

3. the best point to measure the average velocity had to be located in the velocity profile;

Table 4-1. Velocity Profile of the Flow Test in Londonderry Reservoir Flow of 9.73 ML/D

Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.030	0.00	0.0114	0.004
1	25.4	0.147	0.68	0.0214	0.015
2	50.8	0.121	0.53	0.0193	0.010
3	76.2	0.115	0.49	0.0173	0.008
4	101.6	0.117	0.50	0.0153	0.008
5	127.0	0.116	0.50	0.0132	0.007
6	152.4	0.111	0.47	0.0112	0.005
7	177.8	0.113	0.48	0.0092	0.004
8	203.2	0.115	0.49	0.0072	0.004
9	228.6	0.115	0.49	0.0051	0.003
10	254.0	0.105	0.43	0.0031	0.001
11	279.4	0.098	0.39	0.0011	0.000
12	304.8	0.097	0.38	0.0000	0.000
13	330.2	0.101	0.41	0.0011	0.000
14	355.6	0.099	0.40	0.0031	0.001
15	381.0	0.095	0.37	0.0051	0.002
16	406.4	0.093	0.36	0.0072	0.003
17	431.8	0.090	0.35	0.0092	0.003
18	457.2	0.092	0.36	0.0112	0.004
19	482.6	0.095	0.37	0.0132	0.005
20	508.0	0.095	0.37	0.0153	0.006
21	533.4	0.096	0.38	0.0173	0.007
22	558.8	0.082	0.29	0.0193	0.006
23	584.2	0.069	0.22	0.0214	0.005
24	609.6	0.030	0.00	0.0114	0.001
Total				0.2697	**0.111**
Average Velocity (m/s)				**0.41**	

(1) Test date September 17, 1991

(2) Begin at 12:50 AM and end at 13:10 AM

(3) Used MSR Magmeter with 0 - 1 m/s range

(4) MSR Magmeter insertion scale in inches

(5) Flowmeter of the pumping station average = 9.73 ML/D (112.6 l/s)

Table 4-2. Velocity Profile of the Flow Test in Londonderry Reservoir Flow of 20 ML/D

Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.030	0.01	0.0114	0.006
1	25.4	0.214	1.08	0.0214	0.023
2	50.8	0.206	1.04	0.0193	0.020
3	76.2	0.215	1.09	0.0173	0.019
4	101.6	0.215	1.09	0.0153	0.017
5	127.0	0.211	1.06	0.0132	0.014
6	152.4	0.211	1.06	0.0112	0.012
7	177.8	0.210	1.06	0.0092	0.010
8	203.2	0.210	1.06	0.0072	0.008
9	228.6	0.215	1.09	0.0051	0.006
10	254.0	0.198	0.99	0.0031	0.003
11	279.4	0.175	0.85	0.0011	0.001
12	304.8	0.184	0.90	0.0000	0.000
13	330.2	0.191	0.94	0.0011	0.001
14	355.6	0.195	0.97	0.0031	0.003
15	381.0	0.197	0.98	0.0051	0.005
16	406.4	0.187	0.92	0.0072	0.007
17	431.8	0.176	0.86	0.0092	0.008
18	457.2	0.171	0.83	0.0112	0.009
19	482.6	0.180	0.88	0.0132	0.012
20	508.0	0.185	0.91	0.0153	0.014
21	533.4	0.192	0.95	0.0173	0.016
22	558.8	0.165	0.79	0.0193	0.015
23	584.2	0.128	0.57	0.0214	0.012
24	609.6	0.030	0.01	0.0114	0.003
Total				0.2697	**0.243**
Average Velocity (m/s)				**0.90**	

(1) Test date September 17, 1991

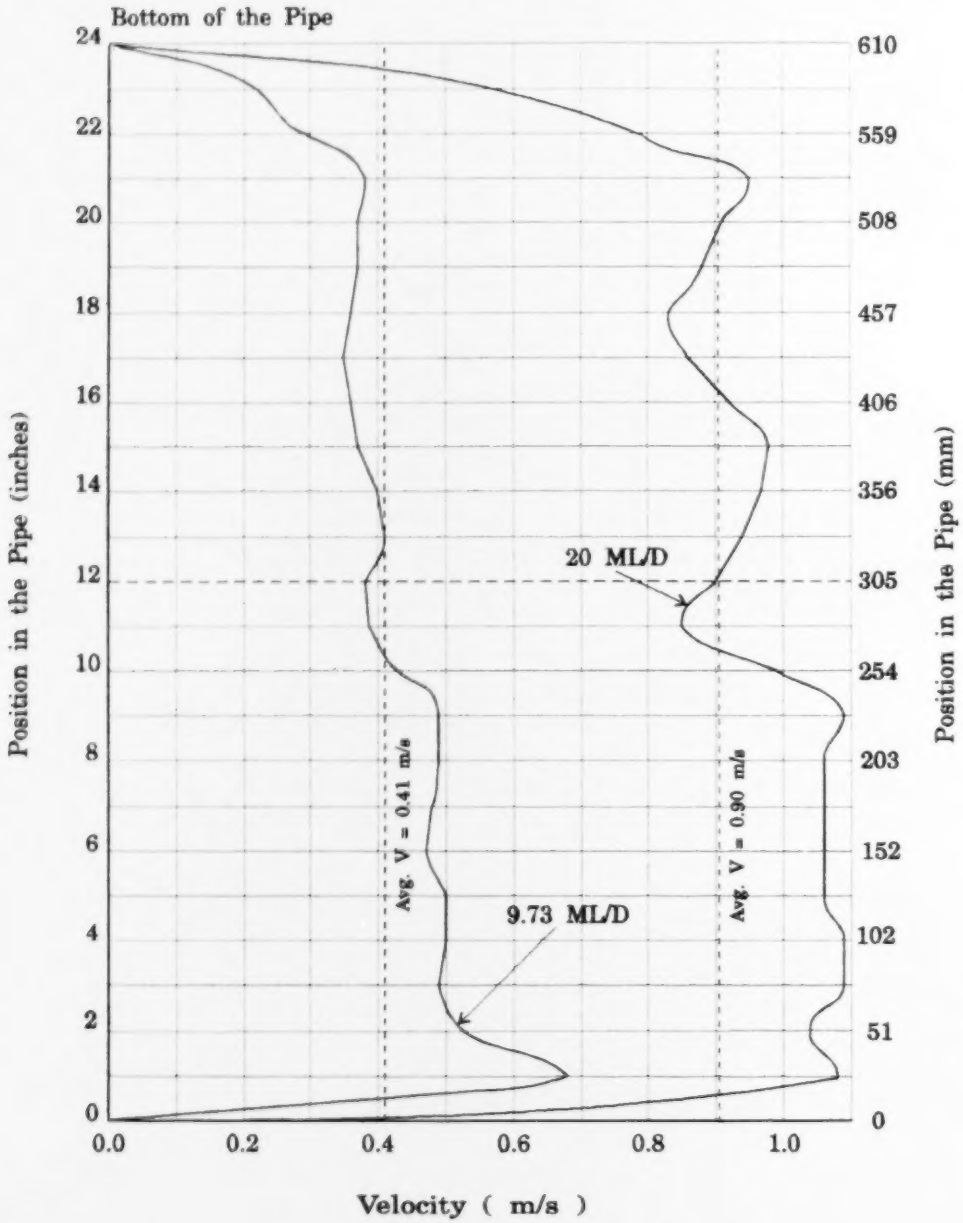
(2) Begin at 14:25 AM and end at 14:50 AM

(3) Used MSR Magmeter with 0 - 1 m/s range

(4) MSR Magmeter insertion scale in inches

(5) Flowmeter of the pumping station average = 20.0 ML/D (231.5 l/s)

Figure 4-1. Velocity Profile of the Flow Test in Londonderry Reservoir



(Test date: September 17, 1991)

4. flow measurement presented good accuracy when defining the average velocity point in the velocity profile, and;

5. flow measurement with a portable magnetic flowmeter kept good accuracy when compared to a permanent installed flowmeter.

4.2. LINE I TEST

Head loss testing in Line I was done from 11:40:00 AM to 14:27:30 PM on September 30, 1991. Initially, this line was scheduled for a 24 hour test. However, to isolate the pipe test section three important valves in the Riverbend area needed to be closed. The branches that were closed were not a problem if closed out of the peak demand period, but closing those branches during peak hours might have created pressure and flow constraints.

Pressure was measured in the chamber of valve No. 25936 (chamber 1), and flow and pressure in the chamber of valve No. 25345 (chamber 2). Each piece of instrument was installed in an isolated air vent valve. The difference in elevation between the two chambers was obtained from the pipeline "as built" drawings. Chamber 2 was 7.40 m higher than chamber 1. The effect of fittings along the section was considered negligible. The equivalent length of those fittings corresponded to 2.4 percent of the total length.

The control valves and the pipe in the chambers were diameter 900 mm, consequently, a diameter smaller than the diameter of the pipeline (1050 mm). Therefore, it was not possible to measure the actual internal diameter of the pipeline. The diameter used in the calculations was the nominal diameter (1050 mm).

Table 4-3 presents the results of the velocity profile and Figure 4-2 shows the plot of the velocity profile. The average velocity was measured in a position corresponding to 33.3 percent of the pipe radius from the top of the pipe (X_4=150 mm, see Figure 3-16). The velocity at this point was 4.2 percent smaller than the average requiring a correction when calculating flow in the pipe.

Data were collected every 15 seconds resulting in 656 observations. Figure 4-3 is a plot of the field test data. Observing those plots, it was possible to infer the following:

- pressure head in chamber 1 showed a wider spread than in chamber 2;
- pressure head in both chambers showed similar behavior during the test period;
- flow rate was kept approximately constant from 11:40 to 12:45 and from 13:45 to 14:27, and;
- flow rate had a peak between 12:45 to 13:45.

Table 4-3. Velocity Profile of Line I

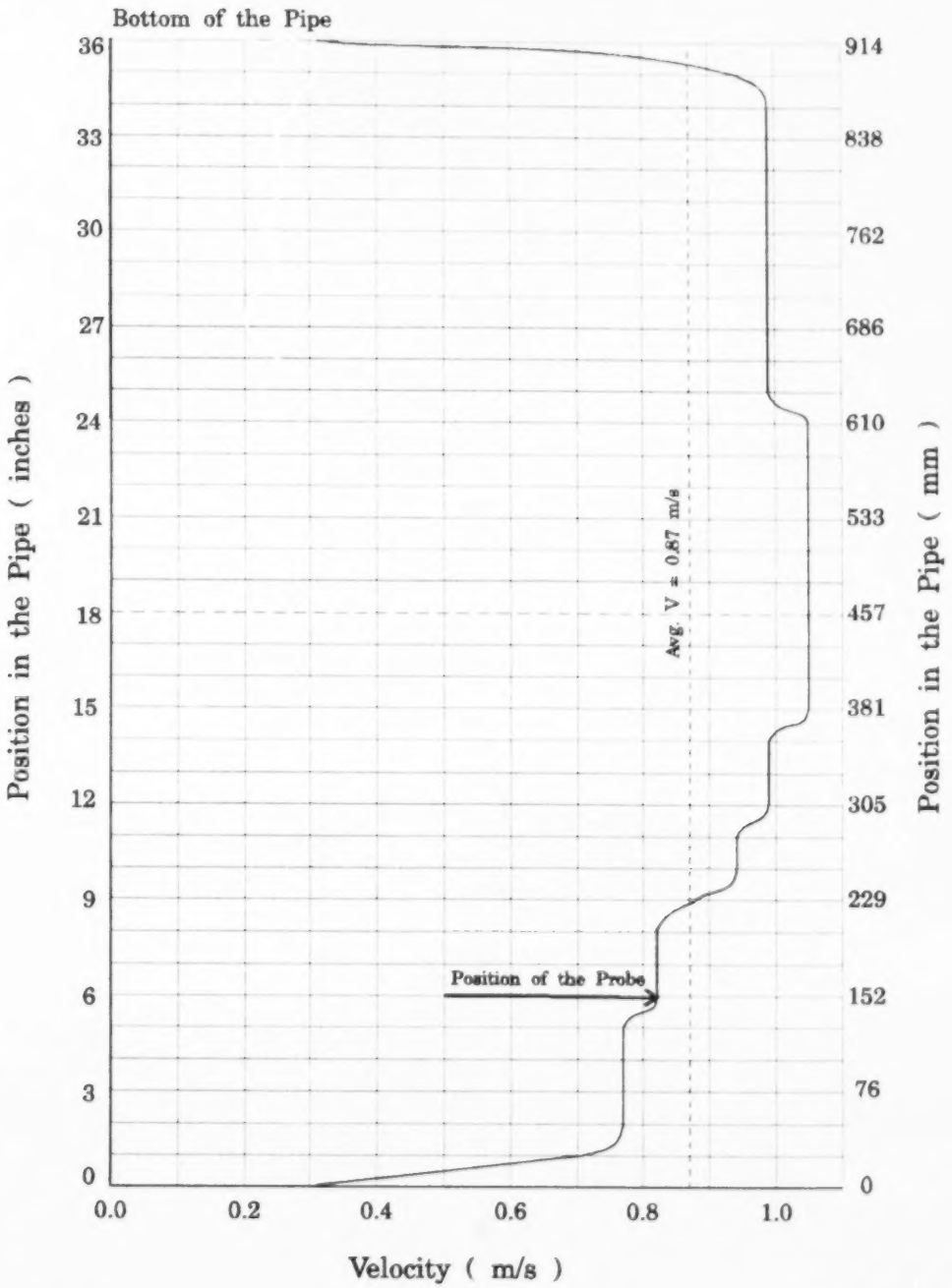
Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.050	0.31	0.0175	0.005
1	25.4	0.085	0.71	0.0335	0.024
2	50.8	0.090	0.77	0.0315	0.024
3	76.2	0.090	0.77	0.0295	0.023
4	101.6	0.090	0.77	0.0274	0.021
5	127.0	0.090	0.77	0.0254	0.019
6	152.4	0.095	0.82	0.0234	0.019
7	177.8	0.095	0.82	0.0214	0.018
8	203.2	0.095	0.82	0.0193	0.016
9	228.6	0.100	0.88	0.0173	0.015
10	254.0	0.105	0.94	0.0153	0.014
11	279.4	0.105	0.94	0.0132	0.012
12	304.8	0.110	0.99	0.0112	0.011
13	330.2	0.110	0.99	0.0092	0.009
14	355.6	0.110	0.99	0.0072	0.007
15	381.0	0.115	1.05	0.0051	0.005
16	406.4	0.115	1.05	0.0031	0.003
17	431.8	0.115	1.05	0.0011	0.001
18	457.2	0.115	1.05	0.0000	0.000
19	482.6	0.115	1.05	0.0011	0.001
20	508.0	0.115	1.05	0.0031	0.003
21	533.4	0.115	1.05	0.0051	0.005
22	558.8	0.115	1.05	0.0072	0.008
23	584.2	0.115	1.05	0.0092	0.010
24	609.6	0.115	1.05	0.0112	0.012
25	635.0	0.110	0.99	0.0132	0.013
26	660.4	0.110	0.99	0.0153	0.015
27	685.8	0.110	0.99	0.0173	0.017
28	711.2	0.110	0.99	0.0193	0.019
29	736.6	0.110	0.99	0.0214	0.021
30	762.0	0.110	0.99	0.0234	0.023
31	787.4	0.110	0.99	0.0254	0.025
32	812.8	0.110	0.99	0.0274	0.027
33	838.2	0.110	0.99	0.0295	0.029
34	863.6	0.110	0.99	0.0315	0.031
35	889.0	0.105	0.94	0.0335	0.031
36	914.4	0.050	0.31	0.0175	0.005
Total				0.6233	**0.541**
Average Velocity (m/s)				**0.87**	

(1) Test date September 30, 1991

(2) Used MSR Magmeter with 0 - 2 m/s range

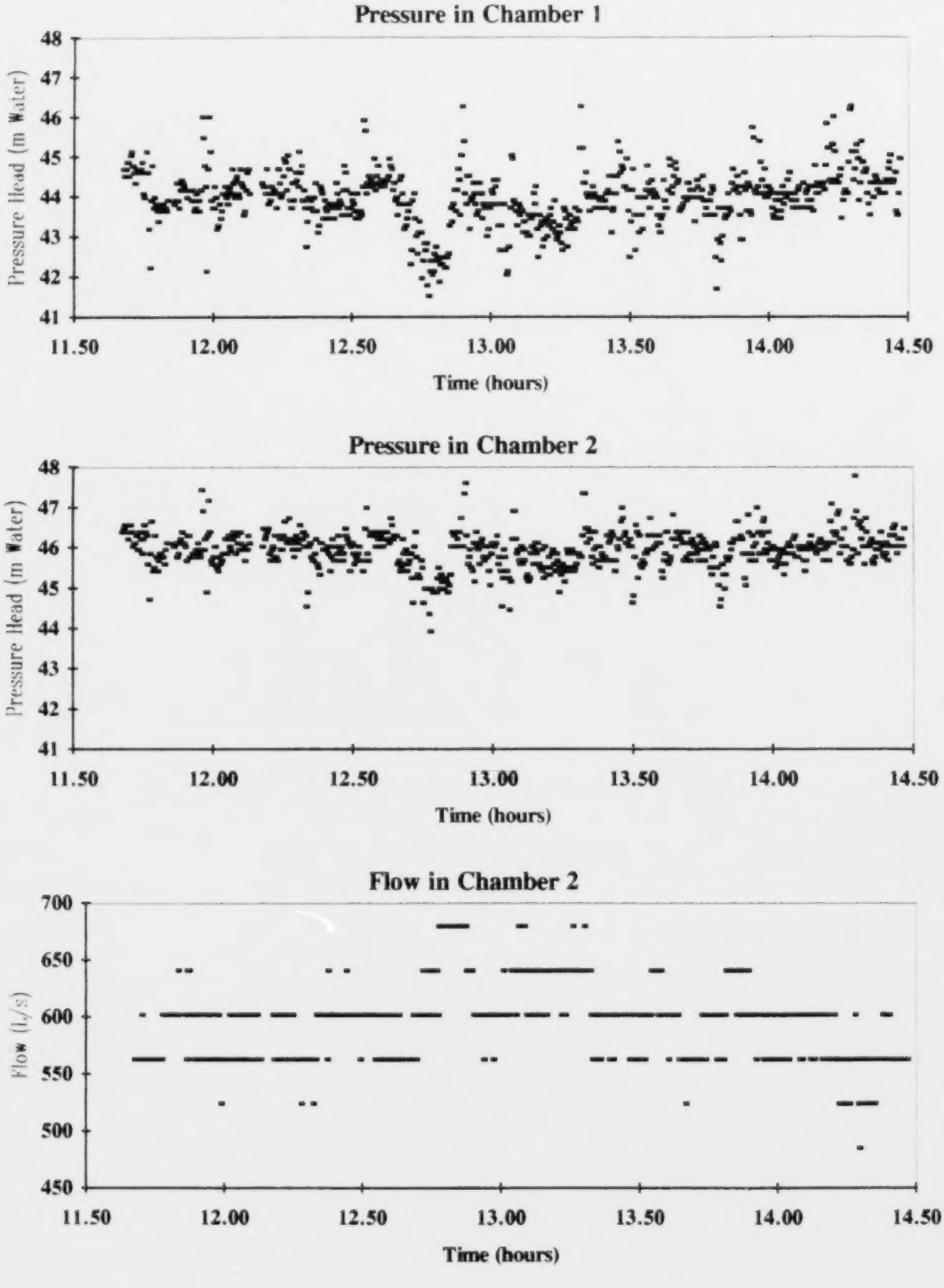
(3) MSR Magmeter insertion scale in inches

Figure 4-2. Velocity Profile for Line I



(Test date: September 30, 1991)

Figure 4-3 . Plot of Data for Line I



Chamber 2 (pressure and flow measurement) was located in a long straight section of pipe, but chamber 1 (pressure measurement) was located close to an intersection in which the pipeline (1050 mm) divided in two other of small diameters. Moreover, both chambers had 1050 mm to 600 mm reducers in every chamber along this main, for the installation of a small diameter gate valve (900 mm). All those appurtenances allowed the creation of eddy currents in the flow. Consequently, generating head losses and a pulsating pattern in the pressure and flow measurements.

The gate valves, which were supposed to be completely open, did not create so much disturbances. However, reducers produced a sudden contraction and expansion in the flow streamlines, therefore, generating many more eddies. Chamber 2 measurements were aggravated due to the close location of the chamber to the point in which the 1050 mm line was divided. In summary, those facts explain the scatter pattern of the measurements in both chambers. Moreover, it also explains the larger spread of the observations in chamber 2.

The average curves of both pressure head followed similar pattern. Some points may be highlighted:

- o the periods that presented steady flow showed an approximately constant pressure head, and;
- o the flow rate increase at 12:45 showed a sharp decrease in pressure head, followed by a stabilization.

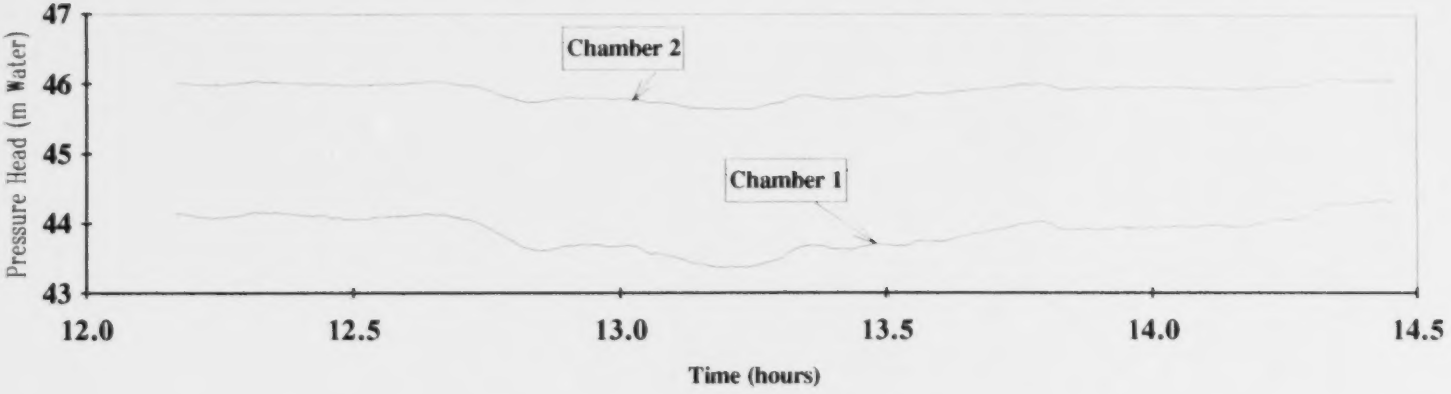
The water distribution system Control Center tries to keep constant the pressure head in the distribution network. However, variations in demand without changing the pumping patterns provokes drops and rises in pressure head. In the head loss test of Line I, flow rate rose at 12:45, followed by a sharp pressure head drop in both chambers. Probably, another constant speed pump was then turned on or a variable speed pump had its speed adjusted, normalizing the pressure in the previous level.

The correlated behavior of pressure head in both chambers and flow rate showed that data were dependent on each other. Therefore, trivial statistics calculations using the sample average and variance were not possible (Berthouex et al, 1978; Box et al, 1978; Berthouex and Hunter, 1981). Berthouex and Hunter (1981) suggested the use of moving average for environmental data interpreting. This method smooths the records, reducing noise and allowing the verification of general trends.

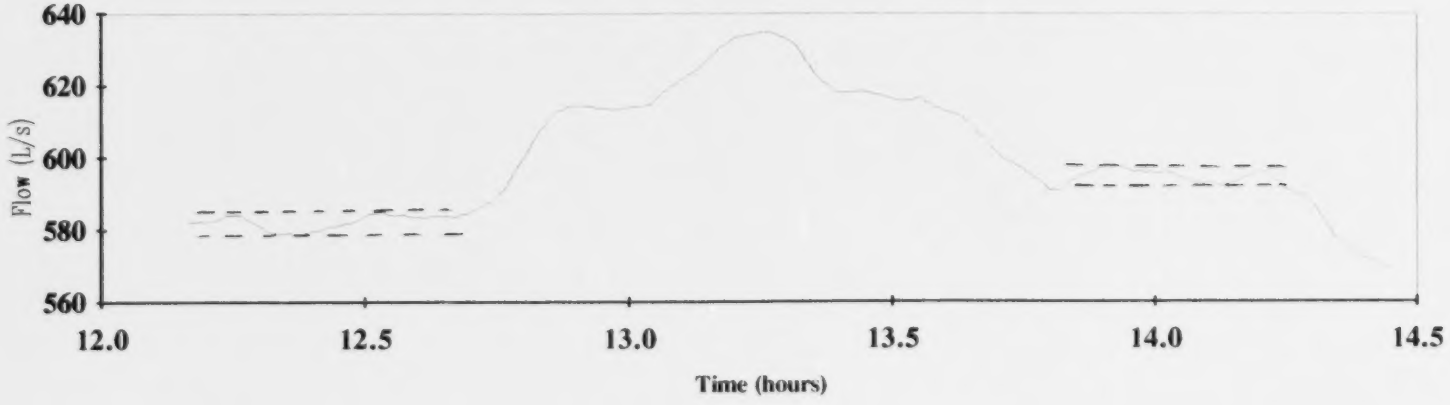
A thirty minute moving average was calculated for each parameter measured (Box and Jenkins, 1976; Berthouex and Hunter, 1981). Figure 4-4 presents the plot of pressure heads and flow moving averages for data of Line I. The analysis of this figure allows fine tuning the conclusions previously drawn, as follows:

- o head loss between chambers was kept constant for each flow rate level;

Figure 4-4. Thirty Minute Moving Average for Data of Line I
Pressure Head in Chambers 1 and 2



Flow in Chamber 2



- o flow rate presented two steady flow levels: from 11:40 to 12:42 and from 13:45 to 14:17.

Calculation of C-factors requires a period of steady flow and constant head loss to obtain good results (Hudson, 1954; Walski, 1984). Therefore, the periods between 12:42 and 13:45 and from 14:17 to the final of the test were not used in C-factor calculation.

The series of observations used in the calculation of the C-factor were considered as a stationary series (Box et al, 1978). Calculated C-factors were statistically dependent, showing a high serial dependence with a lag 1 autocorrelation coefficient ($\delta_1 = 0.96$). This autocorrelation close to 1.0 shows very constant moving average values always close to the total average. Considering the dependence of the series, the variance was corrected as suggested by Box et al (1978). The corrected variance of the average of the dimensionless C-factor was 0.0057.

The average C-factor obtained was 107.3 with the following 95 % confidence interval:

$$107.1 < C < 107.4$$

The small confidence interval shows that the proximity of the isolated C-factor to the average C-factor. Increasing the reliance in the obtained C-factor.

Sharp and Walski (1988) suggested a method to determine the loss of carrying capacity in unlined metallic pipes. This method allows estimation of C-factor for pipes of any age, based on the roughness growth rate and a measured C-factor in a determined year. Therefore, C-factors in function of age were suggested for some commercial large diameter pipes in Table 4-4. The attacking degree of the treated water in Edmonton was classified as moderate attacking to appreciable attacking (Lamont, 1981). Therefore, the growth rate was considered as varying from 0.076 to 0.25 mm/year (Sharp and Walski, 1988).

The following conclusions were drawn from the test results:

1. C-Factors obtained were almost constant for the periods of constant flow;
2. C-Factors obtained were consistent with the values normally used for design: C=100 for old steel unlined pipes in good conditions (Table 2-12);
3. calculated C-Factors lead to the estimate that Edmonton's treated water may have moderate to appreciable attacking quality, and;
4. reducers for valve installation in control valve chambers generated eddies and increased head loss.

Table 4-4. Suggested C Factors for Unlined Steel Pipes Suffering Moderate to Appreciable Water Attack (1)

Pipe Age (years)	C Factor			
	600 mm	750 mm	900 mm	1050 mm
5	123	119	116	115
10	114	113	112	111
15	109	109	109	109
20	105	105	106	106
25	102	103	103	104
30	99	100	101	102
35	96	98	99	100
40	94	96	98	99
45	93	95	96	98
50	91	93	95	96
55	90	92	94	95
60	88	91	93	94
65	87	89	91	93
70	86	88	90	92
75	85	87	89	91
80	84	86	89	90
85	83	86	88	89
90	82	85	87	89
95	81	84	86	88
100	80	83	85	87

(based on Sharp and Waslki, 1988)

(1) adopted roughness growth rate of 0.125 mm/year

4.3. LINE II TEST

Two attempts were made to measure head loss in this line. The first attempt, on September 22, 1991, found the butterfly valve No. 21655 almost totally closed. The second attempt, on October 07, 1991, was tried after the valve was repaired and completely opened.

According to the Maintenance and Operation Department of the Water Branch of the City of Edmonton Public Works, the partially closed valve was in that position for more than two years. The valve was recognized as broken, but the position of the disc was unknown. Water velocity through the valve was higher than 2.0 m/s, generating a high head loss. The Control Center of the Water Branch confirmed the occurrence of problems to feed Londonderry, Rosslyn, and Clareview reservoirs during peak hours. Consequently, much more pumping energy was required to overcome the high head loss, overloading other smaller pipelines. Table 4-5 and Figure 4-5 show the results of the velocity profile obtained with the control valve partially closed.

The second attempt was made using normal flow through the pipeline. However, the definition of the average velocity point was not possible. The tap for flow measurement was too close to the butterfly valve and the probe touched the valve disc close to the center line of the pipe. Therefore, it was not possible to measure the velocity profile. Moreover, the chamber for flow measurement was very close to the junction of Line II with a 1200 mm tunnel, which has a 90° bend coming from a lower to a higher level. This double change in direction created many eddies and a fluctuating pattern when measuring velocities. Attempts to measure point velocity showed a continuous fluctuation of more than 30 percent above and below the average.

In spite of the difficulty in executing the head loss tests, these two attempts allowed some important conclusions to be drawn:

1. field tests are very important in auditing a water distribution system;

2. the position of the chambers to measure flow and pressure are very important for the accuracy of the results in field tests, and;

3. the installation of flow and pressure measurement taps have to be preceded by an analysis of the possible effects of appurtenances and junctions in the results.

4.4. LINE IX TEST

Head loss testing in Line IX was done for 21 hours from 11:02:00 AM on October 9, 1991 to 8:03:00 AM on October 10, 1991. This line was considered as very good for C factor measurement due to its long length. Moreover, the test did not adversely affect water supply, requiring only one 300 mm branch to be closed.

Table 4-5. Velocity Profile of Line II

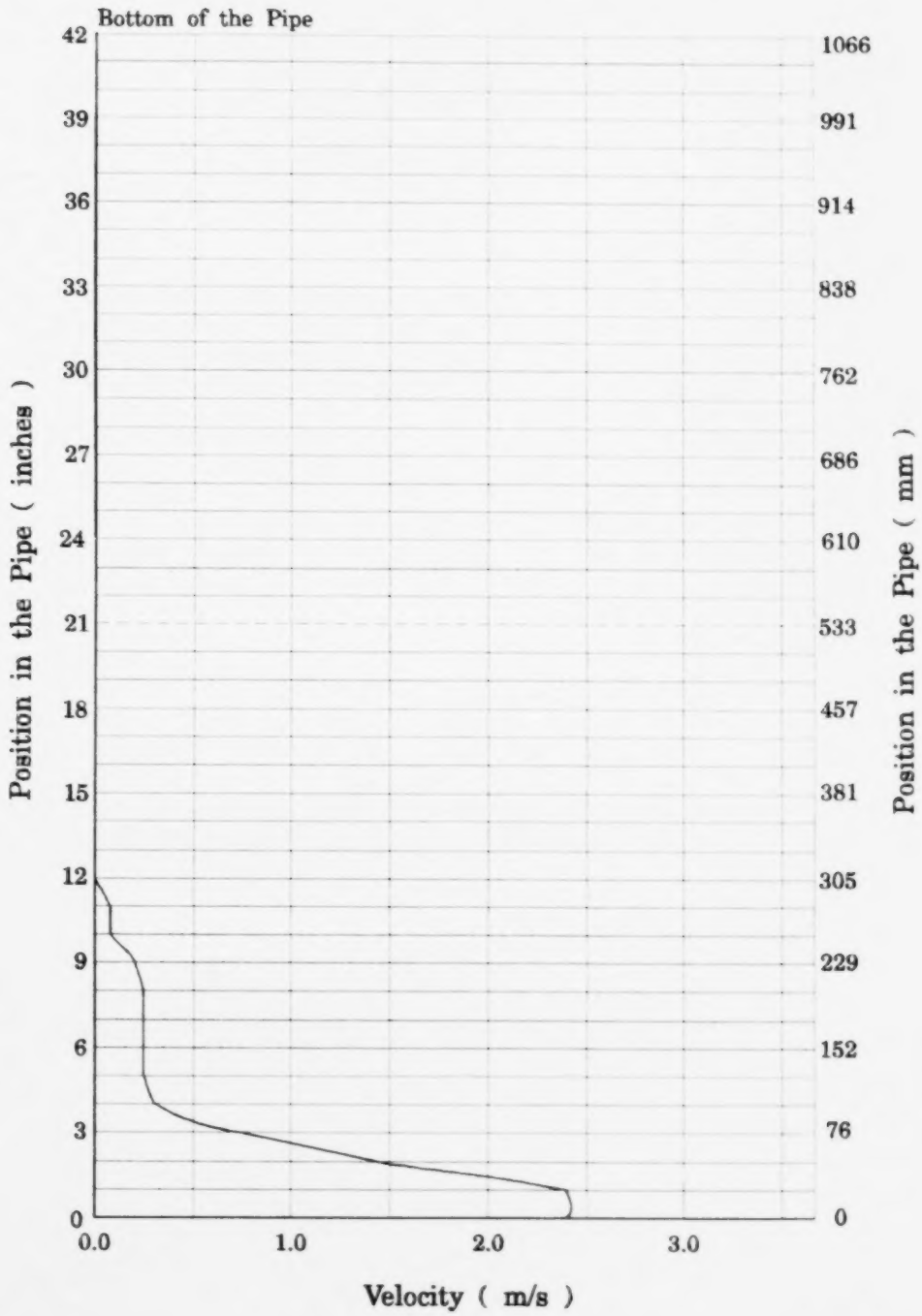
Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.240	2.48	0.0206	0.051
1	25.4	0.240	2.48	0.0396	0.098
2	50.8	0.150	1.45	0.0376	0.054
3	76.2	0.090	0.77	0.0355	0.027
4	101.6	0.050	0.31	0.0335	0.010
5	127.0	0.045	0.25	0.0315	0.008
6	152.4	0.045	0.25	0.0295	0.007
7	177.8	0.045	0.25	0.0274	0.007
8	203.2	0.045	0.25	0.0254	0.006
9	228.6	0.040	0.20	0.0234	0.005
10	254.0	0.030	0.08	0.0214	0.002
11	279.4	0.030	0.08	0.0193	0.002
12	304.8	0.000	0.00	0.0173	0.000
13	330.2	0.000	0.00	0.0153	0.000
14	355.6	0.000	0.00	0.0132	0.000
15	381.0	0.000	0.00	0.0112	0.000
16	406.4	0.000	0.00	0.0092	0.000
17	431.8	0.000	0.00	0.0072	0.000
18	457.2	0.000	0.00	0.0051	0.000
19	482.6	0.000	0.00	0.0031	0.000
20	508.0	0.000	0.00	0.0011	0.000
21	533.4	0.000	0.00	0.0000	0.000
22	558.8	0.000	0.00	0.0011	0.000
23	584.2	0.000	0.00	0.0031	0.000
24	609.6	0.000	0.00	0.0051	0.000
25	635.0	0.000	0.00	0.0072	0.000
26	660.4	0.000	0.00	0.0092	0.000
27	685.8	0.000	0.00	0.0112	0.000
28	711.2	0.000	0.00	0.0132	0.000
29	736.6	0.000	0.00	0.0153	0.000
30	762.0	0.000	0.00	0.0173	0.000
31	787.4	0.000	0.00	0.0193	0.000
32	812.8	0.000	0.00	0.0214	0.000
33	838.2	0.000	0.00	0.0234	0.000
34	863.6	0.000	0.00	0.0254	0.000
35	889.0	0.000	0.00	0.0274	0.000
36	914.4	0.000	0.00	0.0295	0.000
37	939.8	0.000	0.00	0.0315	0.000
38	965.2	0.000	0.00	0.0335	0.000
39	990.6	0.000	0.00	0.0355	0.000
40	1016.0	0.000	0.00	0.0376	0.000
41	1041.4	0.000	0.00	0.0396	0.000
42	1066.8	0.000	0.00	0.0206	0.000
Total				0.8548	**0.278**
Average Velocity (m/s)				**0.32**	

(1) Test date September 22, 1991

(2) Used MSR Magmeter with 0 - 2 m/s range

(3) MSR Magmeter insertion scale in inches

Figure 4-5. Velocity Profile for Line II



(Test date September 22, 1991)

The extension between testing chambers obtained from the "as built" drawings was 3556 m. However, close to Papaschase reservoir and before chamber 2 there was a diameter reduction from 600 to 450 mm with approximately 15 m of 450 mm pipe. Converting this length of 450 mm pipe to equivalent 600 mm pipe, the total length of the test section became 3699 m, which was used in the calculations.

Pressure was measured in the chamber of valve No. 42267 (chamber 1), and flow and pressure in the chamber of the valve No. 42600 (chamber 2). In chamber 2, only one tap with a gate valve was available. Consequently, it was necessary to use a tee to measure flow and pressure. Based on elevations from the pipeline "as built" drawings chamber 2 was 21.134 m higher than chamber 1.

The effect of fittings along the section was considered negligible. Transforming the head loss generated by fittings in equivalent length, it corresponded to 3.0 percent of the total length. The diameter used in the calculations was the nominal diameter (600 mm). The pipe in which flow was measured was 450 mm, preventing the determination of the actual pipeline diameter.

The values that were measured and the plot of the velocity profile are presented in Table 4-6 and Figure 4-6, respectively. The point at which the average velocity was measured was at 50 percent of the pipe radius from the top of the pipe (X_4=112.5 mm, see Figure 3-16). At this point, the velocity was 5.0 percent higher than the average velocity requiring a correction of the flow.

Data were collected every 30 seconds resulting in 2523 observations. Figure 4-7 is a plot of the field test data. Analyzing these plots the following was observed:

- pressure head in chamber 2 presented a much wider spread than in chamber 1;
- pressure head in both chambers showed a similar behavior during the test period;
- flow rate was kept approximately constant from 11:02 to 16:37 (October 9) and from 16:37 (October 9) to 5:12 (October 10), and;
- flow rate had a huge drop between 5:12 to 5:53 (October 10).

The tap in chamber 1 (pressure measurement) was located approximately 5 m downstream a tee for the junction of Line IX (600 mm) with a 900 mm main and a gate valve. Furthermore, downstream this tap there was a 90° elbow. The tap in chamber 2 (pressure and flow measurement) was located between a 90° elbow, a butterfly valve, and a tee, being located approximately 5 m from each fitting. Moreover, the pipe in the last 15 m before the tap was reduced from 600 mm to 450 mm.

The turbulence produced in the flow of chamber 1 was much more due to the presence of the tee, since the gate valve was checked and was completely open. However, flow

Table 4-6. Velocity Profile of Line IX

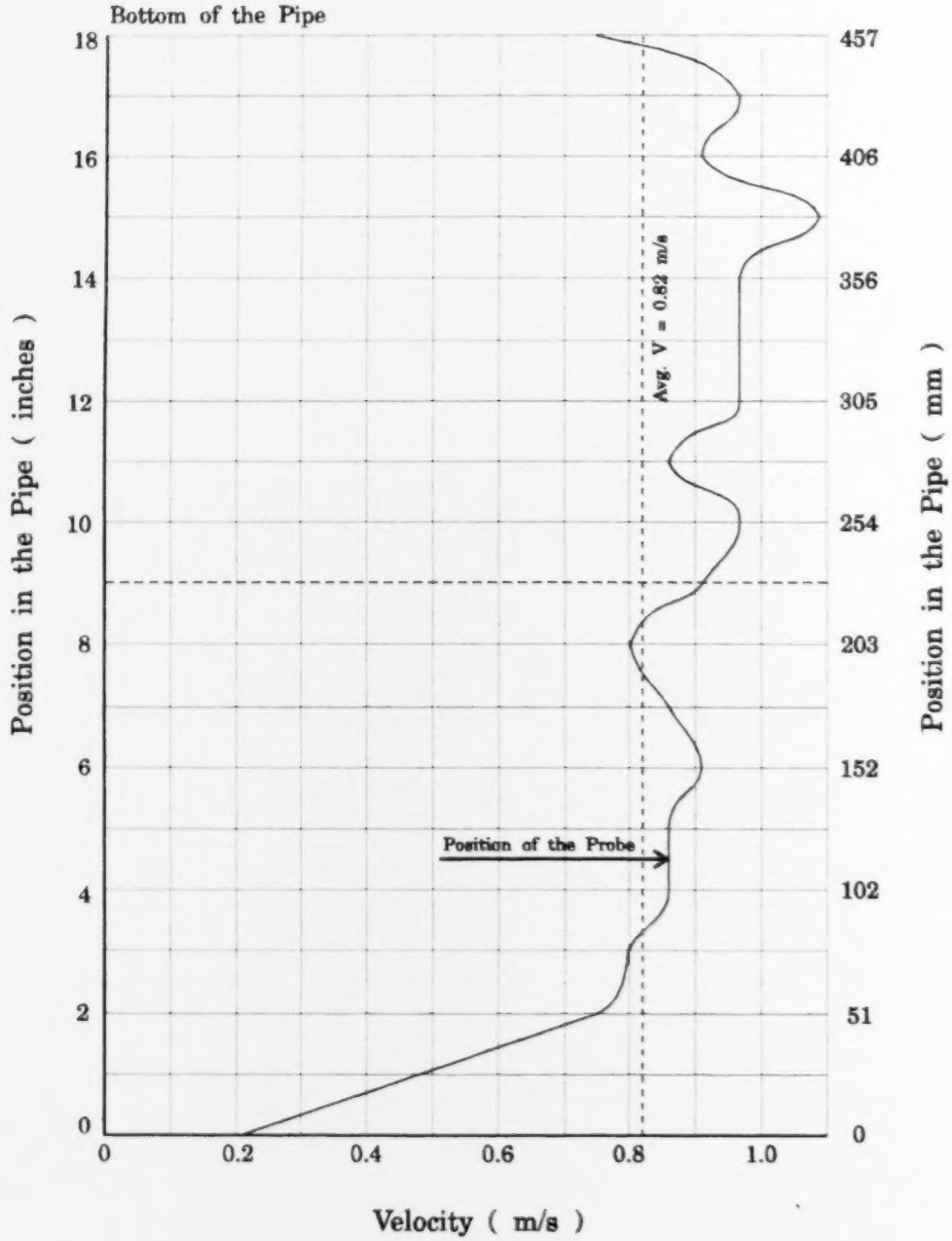
Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.040	0.21	0.0079	0.002
1	25.4	0.065	0.48	0.0153	0.007
2	50.8	0.090	0.75	0.0132	0.010
3	76.2	0.095	0.80	0.0112	0.009
4	101.6	0.100	0.86	0.0092	0.008
5	127.0	0.100	0.86	0.0072	0.006
6	152.4	0.105	0.91	0.0051	0.005
7	177.8	0.100	0.86	0.0031	0.003
8	203.2	0.095	0.80	0.0011	0.001
9	228.6	0.105	0.91	0.0000	0.000
10	254.0	0.110	0.97	0.0011	0.001
11	279.4	0.100	0.86	0.0031	0.003
12	304.8	0.110	0.97	0.0051	0.005
13	330.2	0.110	0.97	0.0072	0.007
14	355.6	0.110	0.97	0.0092	0.009
15	381.0	0.120	1.09	0.0112	0.012
16	406.4	0.105	0.91	0.0132	0.012
17	431.8	0.110	0.97	0.0153	0.015
18	457.2	0.090	0.75	0.0079	0.006
Total				0.1468	**0.120**
Average Velocity (m/s)				**0.82**	

(1) Test date October 9 and 10, 1991

(2) Used MSR Magmeter with 0 -2 m/s range

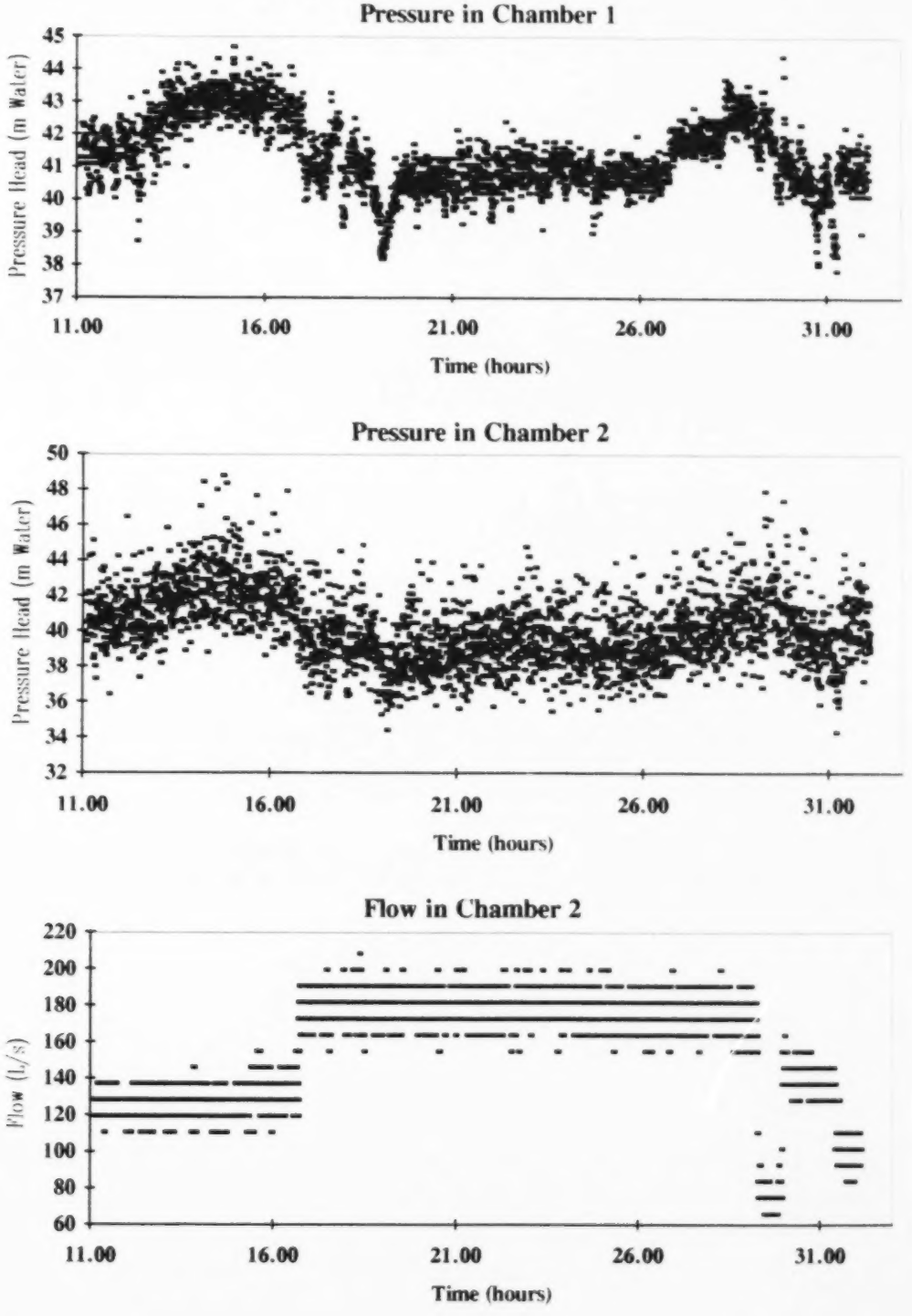
(3) MSR Magmeter insertion scale in inches

Figure 4-6. Velocity Profile for Line IX



(Test date October 9, 1991)

Figure 4-7. Plot of Data for Line IX



in chamber 2 had three appurtenances and a pipe reduction generating turbulence. Therefore, all these changes in flow regime were responsible for the greater spread in data obtained in this chamber.

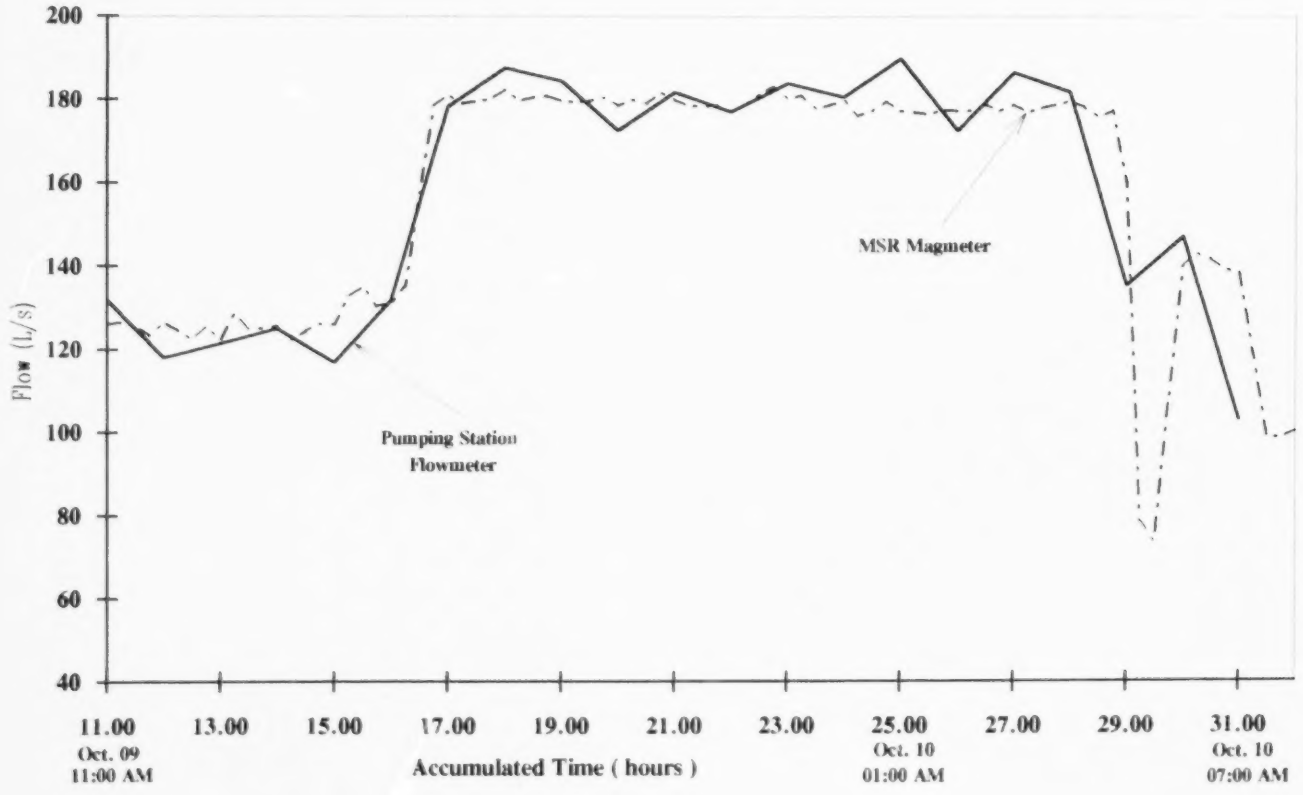
Pressure and flow are continuously monitored at the entrance of Papaschase pumping station, which is approximately 50 m downstream chamber 2. Monitored data is reported hourly by the Water Branch of the City of Edmonton Public Works. Therefore, it was possible to compare flow and pressure measurements from the head loss test with the monitored data. Figure 4-8 shows a plot of both flow rates and Figure 4-9 shows a plot of pressures. Although data from the City of Edmonton was furnished hourly, it was possible to draw some conclusions:

1. flow (Figure 4-8):
    - o both measurements had similar behavior and values;
    - o major differences were in periods between even hours, and;
    - o the maximum differences found in whole hours measurements were approximately $\pm$ 10 % (5:00 on October 9 and 15:00 on October 10).
2. pressure (Figure 4-9):
    - o values measured in Papaschase pumping station by the City of Edmonton fluctuated considerably below and above the values measured in the head loss test, and;
    - o the maximum difference found in whole hours measurements were approximately $\pm$ 13 % (22:00 and 23:00 on October 9).

The maximum differences found in the flow measurement values were considered acceptable according to error criteria cited by Hinds (1946) of an acceptance of $\pm$ 10 % error. Figure 4-8 shows that most of the time the values were closer, except the period from 18:00 to 19:00 on October, which had a huge drop in flow rate in the head loss tests, not shown in the monitored data. This drop happened exactly between two whole hours when monitored flow data was not furnished, consequently, omitting the flow drop. In summary, the comparison showed a reasonable accordance between values.

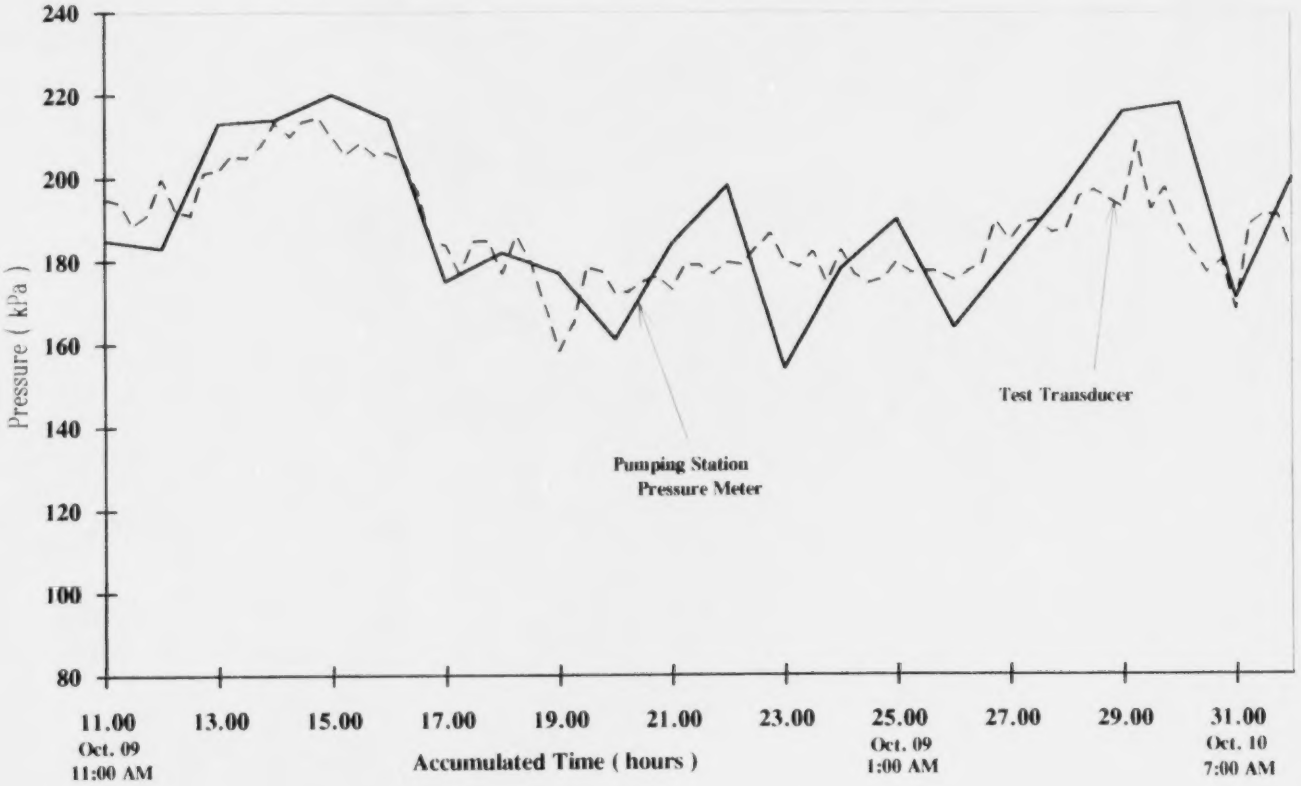
Pressure measurement in the entrance of the Papaschase pumping station is done using a pressure transducer, therefore, using the same method used for pressure measurement in the field tests. The point in which pressure is measured in the pumping station is so affected or more by turbulence generated by appurtenances as the point used for pressure measurement in chamber 2. Figure 4-7 shows that in that situation values for isolated observations fluctuate considerably below and above the average, not allowing accurate measurement of point readings. Therefore, the comparison of the values

Figure 4-8. Line IX: Flow Measured with MSR Magmeter and Flowmeter in the Pumping Station



(Test beginning in October 09, 1991 at 11:02 AM and finishing October 10,1991 at 08:03 AM)

Figure 4-9. Line IX: Pressure Measured with Test Transducer and Pressure Meter in the Pumping Station



(Test beginning in October 09, 1991 at 11:02 AM and finishing October 10,1991 at 08:03 AM)

obtained every 30 seconds with hourly measured values was not adequate. However, it showed that some attention must be paid when analyzing data summaries for pressures based on hourly point observations instead of hourly averages from shorter periods observations.

Although not as evident as in Line I, the behavior of pressure heads and flow in Line IX showed a dependent relationship with similar waving. Therefore, the same statistical data treatment applied to Line I was applied for smoothing the records and reducing noise in Line IX (Berthouex et al, 1978; Box et al, 1978; Berthouex and Hunter, 1981). Figure 4-10 shows the plot of a thirty minute moving average for data of Line IX. Analyzing this figure, the following was observed:

- pressure at the point in which flow rate was sharply reduced was approximately equal in both chambers (5:15 on October 10), and;
- measured pressure head in chamber 2 at this point was sometimes higher than in chamber 1.
- head loss was kept constant for the period of constant flow between 17:00 on October 9 to 5:10 on October 10;

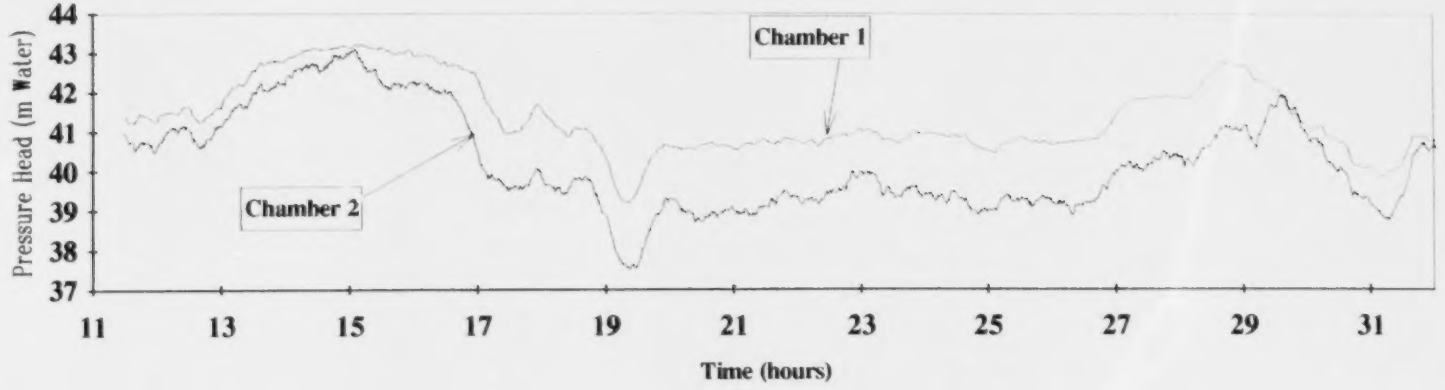
At 17:00 on October 10, 1991, the flow rate was reduced from an average of 180 L/s to 75 L/s. This sharp reduction provoked an abnormal behavior of the measured pressure head, showing zero or negative head losses between chambers 1 and 2. Both cases were not acceptable. The first because there was some flow still passing through the pipeline, consequently, not allowing zero head loss. The second case is not practically viable.

Analyzing the occurrence, three explanations may be suggested. First was due to error of the pressure meters, which showed poor accuracy for low head losses. Second, the flow in the fill line is controlled by operating a telemetering operated butterfly valve at the entrance of the reservoir. Quick and sharp valve operation, as done during the tests, many generate a hydraulic transient. The wave created by the transient may have increased the pressure in chamber 2, not affecting chamber 1, which was almost 4 km upstream.

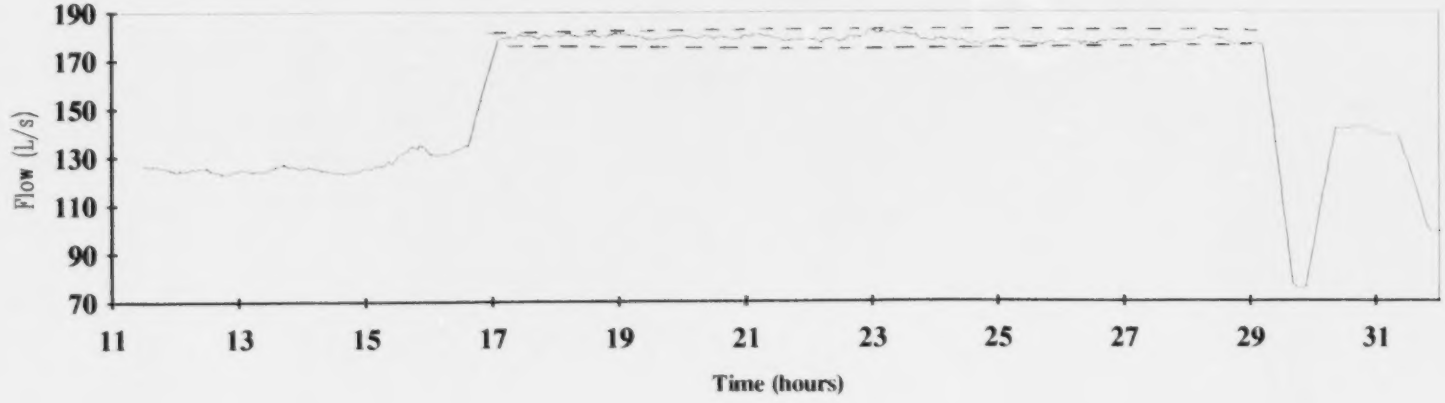
C-factor was calculated for the long period of steady flow between 17:00 on October 9 to 5:10 on October 10. Other periods were not used because head loss patterns or flow rates were considered not constant.

The series of observations for C-factor calculation were considered as stationary series (Box et al, 1978). Calculated C-factors were statistically dependent, showing a high serial dependence with a lag 1 autocorrelation coefficient ($\delta_1 = 0.98$). This autocorrelation close to 1.0 shows very constant moving average values always close to the total average. Considering the dependence of the series, the variance was corrected as suggested by Box et al (1978). The corrected variance of the average of the dimensionless C-factor was 0.27.

Figure 4-10. Thirty Minute Moving Averages for Data of Line IX
Pressure Head in Chambers 1 and 2



Flow in Chamber 2



The average C-factor obtained was 165.8 with the following 95 % confidence interval:

$$164.8 < C < 166.8$$

The obtained C-factor may be considered adequate for a straight section of pipe, presenting a small head loss due to appurtenances. A C factor of 166 was obtained by Irwin (1946) in tests developed in pressure concrete pipelines in Chicago.

The test resulted in the following conclusions:

1. the pressure concrete pipe tested presented a smooth flow pattern with C-factors always above 150;

2. head loss tests in pipes with low velocity required very long pipe test sections to obtain an accurate and measurable head loss;

3. pressure and flow measured in a chamber in which the turbulence generated by appurtenances is high presented considerable fluctuation of readings;

4. point readings for parameters, specially pressure, are not trustful due to readings fluctuation, and;

5. any parameters to be used in head loss and flow tests have to be based on an average of a large number of observations.

4.5. LINE XII TEST

Head loss testing in Line XII was done for approximately 22 hours from 9:55:00 AM on October 01, 1991 to 08:09:30 AM on October 02, 1991. Two tests were performed on that line: one on September 27, 1991 and another on October 01 and 02, 1991. The results of the first test had to be abandoned due to a failure in the data logger of the initial chamber (pressure measurement only). However, the results of the second test were successful. This line was initially considered as one of the best for C factor measurement due to its long length. However, the head loss was often extremely small due to the low velocity in the line.

Pressure was measured in the chamber of valve No. 36357 (chamber 1), and flow and pressure in the chamber of the valve No. 36345 (chamber 2). In this last chamber, there was only one air vent, therefore, the flowmeter and the pressure transducer were installed using a tee. The difference of elevation between the chambers was obtained by surveying, as there was no "as built" drawing available. Chamber 2 was 7.807 m higher than chamber 1.

The pipe test section is very straight, consequently, the effect of fittings along the section was considered very small. Transforming the fitting head losses in equivalent length, the equivalent length was less than 2 percent of the total length.

The material of the pipe inside the valve chamber (steel) was different from the material of pipeline (concrete). Therefore, the diameter of the pipe to be used in calculations was the nominal diameter (1050 mm).

The position of the average velocity in the velocity profile corresponded to 48 percent of the radius from the top of the pipe (X_4=250 mm, see Figure 3-16). Table 4-7 shows the results of the velocity profile measurement and Figure 4-11 shows the plot of the velocity profile.

Test data in the head loss test was measured every 30 seconds obtaining 2718 observations. Figure 4-12 shows the plot of the field test data. The observation of these plots allowed to infer the following:

- the spread of pressure head in both chambers was much smaller than the other two tests;
- pressure head in both chambers showed similar behavior during the test period;
- flow rate was kept constant from 9:55 to 20:36 on October 1, changing constantly after 20:36;
- flow rates were always very low, presenting very low velocities;
- flow rate peak was at midnight, and;
- flow rate had a huge and sharp drop between 2:14 to 3:32 on October 2.

The chambers in which pressure and flow measurements were done were located in a long straight section of pipeline without any fittings to create turbulence. Each valve had only the butterfly valve that was checked as completely open. This fact justifies the much smaller fluctuation of the observations in Line XII comparing to Lines I and IX.

The same statistical data treatment applied to Lines I and IX was applied for smoothing the records and reducing noise in Line XII (Berthouex et al, 1978; Box et al, 1978; Berthouex and Hunter, 1981). Figure 4-13 shows the plot of a thirty minute moving average for data of Line IX. Analyzing this figure, the following was observed:

- head loss was extremely low during the whole head loss test, and;
- period that generated a larger head loss had unsteady flow (11:44 on October 1 to 0:54 on October 2).

Low flows during the test period decreased substantially the accuracy of the head loss test. Therefore, the results obtained were considered not suitable for any C-factor measurement.

Table 4-7. Velocity Profile of Line XII

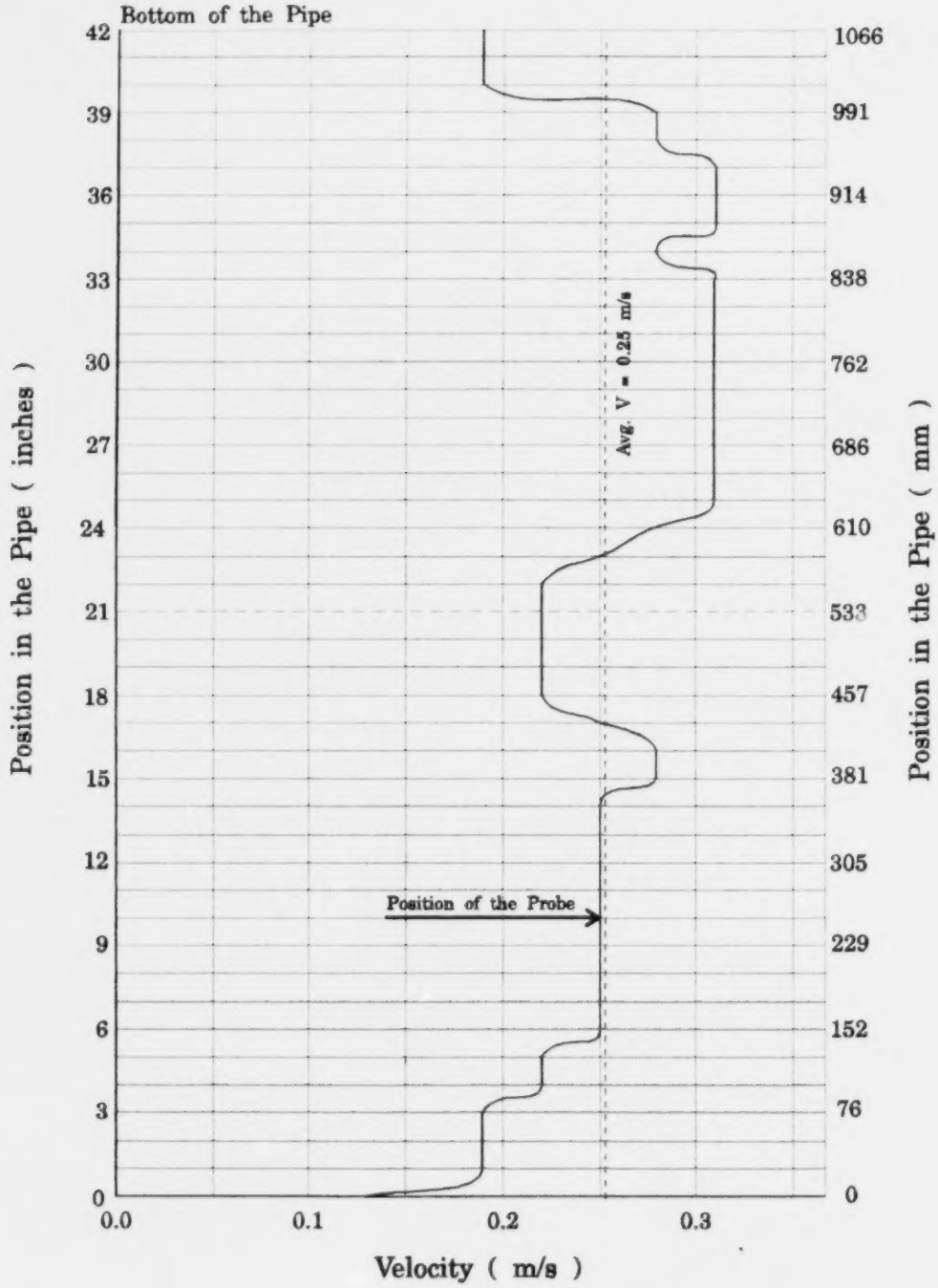
Position		Voltage	Velocity	Anular Area	Flow
(inches)	(mm)	(V)	(m/s)	(m^2)	(m^3/s)
0	0.0	0.055	0.13	0.0206	0.003
1	25.4	0.065	0.19	0.0396	0.008
2	50.8	0.065	0.19	0.0376	0.007
3	76.2	0.065	0.19	0.0355	0.007
4	101.6	0.070	0.22	0.0335	0.007
5	127.0	0.070	0.22	0.0315	0.007
6	152.4	0.075	0.25	0.0295	0.007
7	177.8	0.075	0.25	0.0274	0.007
8	203.2	0.075	0.25	0.0254	0.006
9	228.6	0.075	0.25	0.0234	0.006
10	254.0	0.075	0.25	0.0214	0.005
11	279.4	0.075	0.25	0.0193	0.005
12	304.8	0.075	0.25	0.0173	0.004
13	330.2	0.075	0.25	0.0153	0.004
14	355.6	0.075	0.25	0.0132	0.003
15	381.0	0.080	0.28	0.0112	0.003
16	406.4	0.080	0.28	0.0092	0.003
17	431.8	0.075	0.25	0.0072	0.002
18	457.2	0.070	0.22	0.0051	0.001
19	482.6	0.070	0.22	0.0031	0.001
20	508.0	0.070	0.22	0.0011	0.000
21	533.4	0.070	0.22	0.0000	0.000
22	558.8	0.070	0.22	0.0011	0.000
23	584.2	0.075	0.25	0.0031	0.001
24	609.6	0.080	0.28	0.0051	0.001
25	635.0	0.085	0.31	0.0072	0.002
26	660.4	0.085	0.31	0.0092	0.003
27	685.8	0.085	0.31	0.0112	0.003
28	711.2	0.085	0.31	0.0132	0.004
29	736.6	0.085	0.31	0.0153	0.005
30	762.0	0.085	0.31	0.0173	0.005
31	787.4	0.085	0.31	0.0193	0.006
32	812.8	0.085	0.31	0.0214	0.007
33	838.2	0.085	0.31	0.0234	0.007
34	863.6	0.080	0.28	0.0254	0.007
35	889.0	0.085	0.31	0.0274	0.009
36	914.4	0.085	0.31	0.0295	0.009
37	939.8	0.085	0.31	0.0315	0.010
38	965.2	0.080	0.28	0.0335	0.009
39	990.6	0.080	0.28	0.0355	0.010
40	1016.0	0.065	0.19	0.0376	0.007
41	1041.4	0.065	0.19	0.0396	0.008
42	1066.8	0.065	0.19	0.0206	0.004
Total				0.8548	**0.216**
Average Velocity (m/s)				**0.25**	

(1) Test date October 01 and 02, 1991

(2) Used MSRMagmeter with 0 - 1 m/s range

(3) MSR Magmeter insertion scale in inches

Figure 4-11. Velocity Profile for Line XII



(Test date October 01, 1991)

Figure 4-12. Plot of Data for Line XII

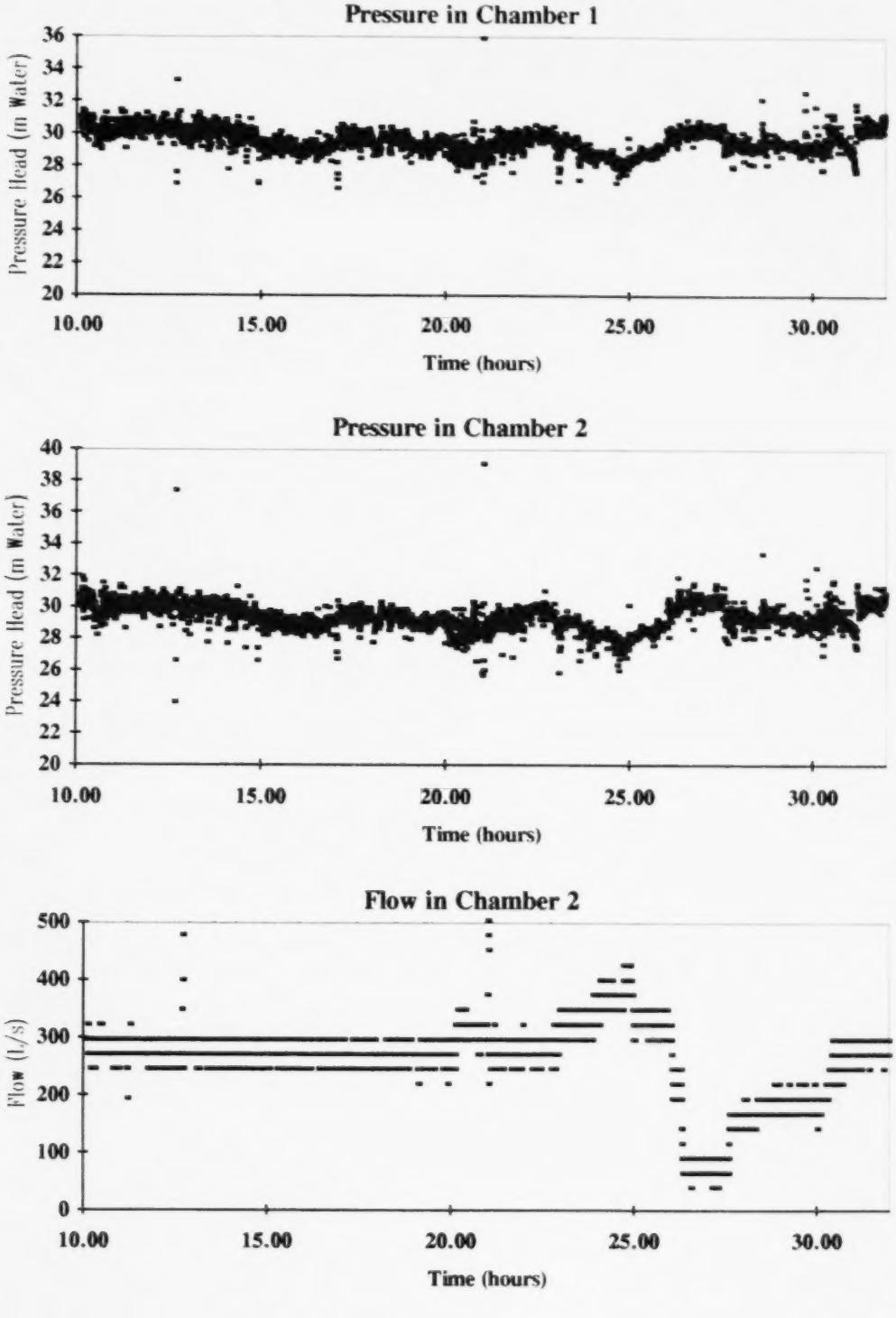
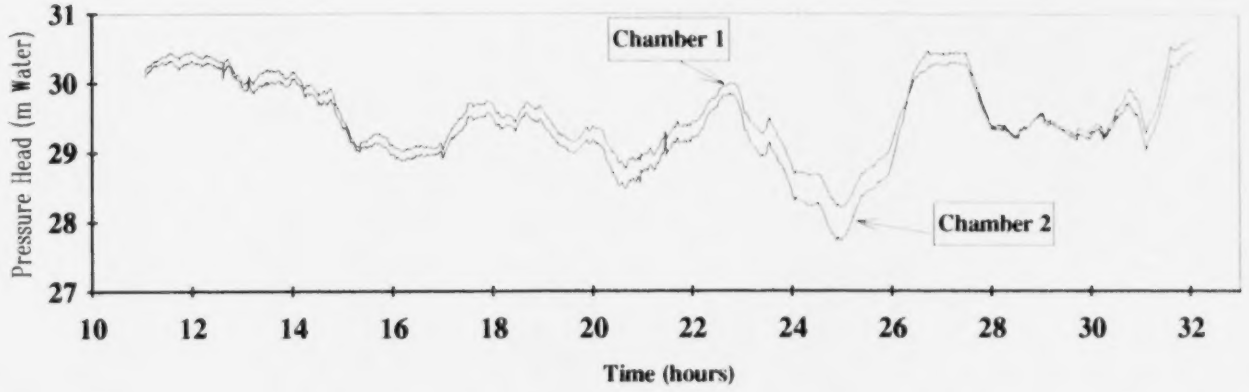
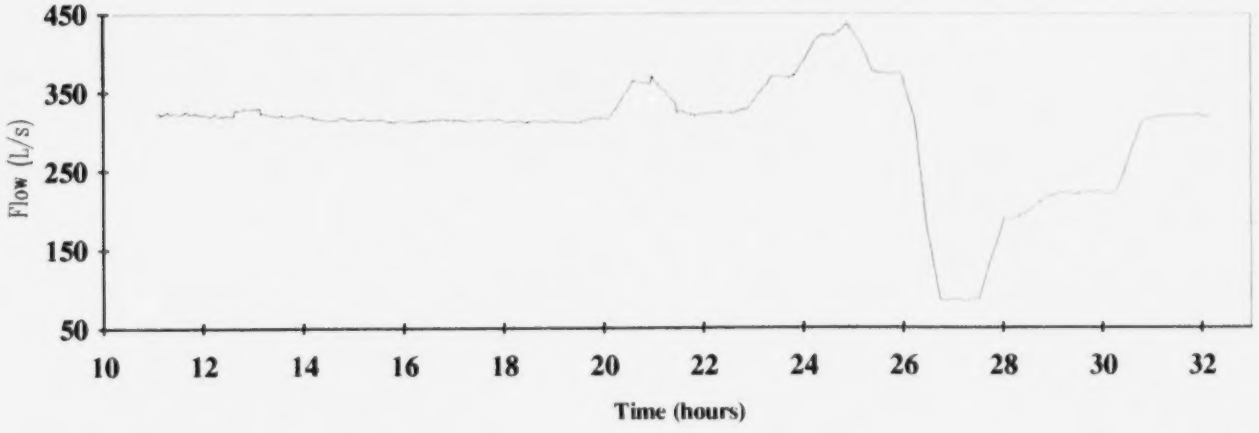


Figure 4-13. Thirty Minute Moving Average for Data of Line XII
Pressure Head in Chambers 1 and 2



Flow in Chamber 2



Head loss test was developed for almost 24 hours, covering the peak demand and low demand periods. Furthermore, the flow was increased on purpose to improve the C-factor measurements. However, it is possible to verify that even at its maximum flow, the main is operating at flow rates extremely below values considered economically good. For an average velocity of 1.2 m/s, considered as an economical velocity for an average demand period, the flow should have to be approximately 1,000 l/s, consequently more than twice the maximum value observed.

The following conclusions were drawn:

1. Line XII was operating much below the velocity considered as economical for a water main;

2. calculation of C-factors was considered inaccurate for very low head losses due to pressure fluctuation, and;

3. pipe test sections longer than that used were necessary to obtain a more accurate head loss.

4.6. PRACTICAL IMPORTANCE OF THE RESULTS

The C-factors obtained from head loss tests in Line I and IX and the facts verified in Lines II and XII, allowed to draw practical conclusions in the water distribution system of the City of Edmonton. Those conclusions covered various areas as simulation modelling, pipeline and pumping station design, pipeline maintenance and operation, and pipeline testing.

Simulation modelling

The results of the field tests showed that the C-factors for metallic pipelines (C=107.3) were in close agreement with the values suggested in Tables 2-5 and 2-12. The C-factor obtained for pressure concrete pipe in Line IX showed that this pipeline was operating with smooth flow regime. Moreover, the obtained C-factor was even higher than C-factors normally used in pipeline design (C=165.8). Considering that Line IX was very straight and almost without fittings, it may be considered to be in favor of safety to use a C-factor of 150 as suggested by manufacturers and technical literature (Table 2-12).

Using the values of Table 2-12 was possible to establish a comparison between those C-factors and C-factors used at the Water Branch of the City of Edmonton simulation model. Table 4-8 shows the principal characteristics of the simulation model pipelines, and establishes a comparison between C-factors. C-factors used in the simulation model were classified as reasonable for values agreeing with Table 2-12, and low, very low and high for values discording from Table 2-12. Figure 4-14 illustrates the C-factor classification in the water distribution network. Table 4-9 quantifies the number of pipes according to C-factor classification.

Table 4-8. C Factors of Edmonton's Water Distribution Network Model

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
1	1	2	1125	1300	80	1950	Concrete	Very Low	140
2	1	3	400	2200	80	1949	Welded Steel	Reasonable	80
3	1	5	821 (1)	2100	80	1962	Steel	Reasonable	80
4	1	6	671 (1)	1800	110	1923	Cast Iron	High	80
5	1	7	900	1000	80	1960	Steel	Reasonable	80
6	1	146	1050	1250	75	1958	Concrete	Very Low	140
7	1	149	1200	2700	80	1969	Concrete	Very Low	140
8	2	3	750	1300	80	1968	Concrete	Very Low	140
9	2	11	400	1700	80	1950	Welded Steel	Reasonable	80
10	2	19	932 (1)	2000	80	1967	Concrete	Very Low	140
11	3	114	400	1300	80	1951	Welded Steel	Reasonable	80
12	3	145	208 (1)	3200	110	1959	Unknown	Probably Reas.	110
13	4	145	750	600	80	1960	Steel	Reasonable	80
14	4	146	1050	1300	75	1958	Concrete	Very Low	140
15	4	147	900	950	110	1958	Lined Steel	Reasonable	110
16	5	6	600	900	100	1958	Steel	High	80
17	5	148	750	700	80	1950	Steel	Reasonable	80
18	6	149	450	1100	100	1909	Cast Iron	High	80
19	6	150	813 (1)	600	120	1961	Steel	High	80
20	7	8	750	900	80	1969	Steel	Reasonable	80
21	7	11	750	1800	80	1960	Steel	Reasonable	80
22	8	9	500	1350	80	1951	Welded Steel	Reasonable	80
23	8	111	750	4500	100	1971	Steel	Reasonable	100
24	9	10	400	1500	80	1957	Steel	Reasonable	80
25	9	17	400	2600	100	1957	Cast Iron	Reasonable	100
26	9	111	300	550	100	1950	Cast Iron	High	80
27	9	113	300	1600	80	1951	Cast Iron	Reasonable	80
28	10	11	794 (1)	1700	80	1962/79	Steel	Reasonable	80
29	10	18	300	1400	120	1955	Asbestos Cement	Reasonable	140
30	10	113	785 (1)	800	120	1960	Steel	High	100
31	12	13	647 (1)	900	110	1951	Welded Steel	High	80
32	12	21	514 (1)	3300	120	1951	Cast Iron/Welded Stl	High	80
33	12	147	900	850	110	1958	Lined Steel	Reasonable	110
34	12	148	395 (1)	1900	110	1930	Cast Iron	High	80
35	13	14	600	1900	110	1951	Steel	High	80
36	13	24	500	1100	90	1950	Cast Iron	Reasonable	80
37	13	41	600	2700	80	1961	Steel	Reasonable	80
38	13	136	320 (1)	2600	120	1955	Cast Iron/Welded Stl	High	80
39	14	25	600	1200	80	1959	Steel	Reasonable	80
40	14	148	750	1100	80	1950	Steel	Reasonable	80
41	15	16	600	900	120	1959	Steel	High	100
42	15	26	1050	1500	80	1969	Concrete	Very Low	140
43	15	148	600	1300	80	1958	Steel	Reasonable	80
44	15	149	1200	300	80	1969	Concrete	Very Low	140
45	15	150	450	750	100	1974	Asbestos Cement	Low	140

(table continued on next page)

Table 4-8. Continued

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
46	16	27	524 (1)	3700	110	1932/49	Cast Iron/Steel	High	80
47	16	150	500	1350	120	1932	Cast Iron	High	80
48	16	151	300	1700	100	1912	Cast Iron	High	80
49	17	111	750	2500	100	1971	Steel	Reasonable	100
50	17	171	812 (1)	1200	110	1962/70	Asb.Cement/Concrete	Low	140
51	17	176	400	500	120	1962	Asbestos Cement	Reasonable	140
52	18	31	750	2300	130	1960	Concrete	Low	140
53	18	113	750	900	100	1960	Steel	Reasonable	100
54	18	182	500	3200	120	1955	Welded Steel	High	80
55	19	35	900	1300	120	1967	Concrete	Low	140
56	19	115	400	1200	80	1951	Welded Steel	Reasonable	80
57	19	182	400	750	120	1949	Welded Steel	High	80
58	20	21	300	3300	110	1951	Cast Iron	High	80
59	20	37	750	2000	125	1964	Concrete	Low	140
60	20	126	300	1850	120	1959	Asbestos Cement	Reasonable	140
61	20	135	300	2400	110	1957	Cast Iron	High	100
62	20	145	750	3350	80	1957/60	Steel	Reasonable	80
63	21	135	300	1000	110	1954	Cast Iron	High	80
64	21	136	300	1700	120	1954	Cast Iron	High	80
65	22	39	300	1950	100	1975	Asbestos Cement	Low	140
66	22	40	643 (1)	600	120	1975	Steel	High	100
67	22	136	450	700	120	1967	Asbestos Cement	Reasonable	140
68	22	190	750	50	50	1976	Steel	Very Low	100
69	24	41	450	1450	80	1951	Cast Iron	Reasonable	80
70	24	137	394 (1)	2800	110	1954/57	Asb. Cem./Weld. Stl	Reasonable	110
71	25	26	381 (1)	800	90	1951	Steel	Reasonable	90
72	25	42	500	1400	80	1951	Welded Steel	Reasonable	80
73	26	43	1050	850	80	1970	Concrete	Very Low	140
74	26	149	342 (1)	1600	90	1909	Cast Iron	High	80
75	26	151	350	650	100	1912	Cast Iron	High	80
76	27	45	500	750	100	1949	Welded Steel	High	80
77	27	152	300	1500	110	1957	Cast Iron	High	80
78	27	193	367 (1)	1800	130	1956	Asb. Cem./Weld. Stl	High	100
79	28	171	750	1600	110	1970	Concrete	Low	140
80	30	116	664 (1)	1550	110	1975	Steel	Reasonable	110
81	30	183	600	1500	110	1982	Concrete	Low	140
82	32	116	750	800	110	1975	Steel	Reasonable	110
83	32	184	300	1000	120	1974	Asbestos Cement	Reasonable	140
84	34	33	450	2600	140	1963	Asbestos Cement	Reasonable	140
85	34	35	300	700	140	1960	Asbestos Cement	Reasonable	140
86	34	51	552 (1)	2250	130	1966/71	Asbestos Cement	Reasonable	140
87	34	182	300	1300	120	1959	Cast Iron	High	100
88	35	36	300	3100	110	1960	Asbestos Cement	Low	140
89	35	52	300	2200	120	1959	Asbestos Cement	Reasonable	140
90	35	123	900	1700	100	1970	Steel	Reasonable	100

(table continued on next page)

Table 4-8. Continued

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
91	36	70	300	2500	90	1964	Asbestos Cement	Low	140
92	36	122	387 (1)	1500	90	1961	Asbestos Cement	Low	140
93	36	181	350	700	110	1961	Asbestos Cement	Low	140
94	37	38	400	2800	125	1963/67	Asbestos Cement	Reasonable	140
95	37	99	750	800	125	1969	Concrete	Low	140
96	37	126	325 (1)	2900	120	1962/67	Asbestos Cement	Reasonable	140
97	38	101	324 (1)	1750	120	1976	PVC/Asb. Cement	Low	140
98	38	131	490 (1)	4100	110	1975/76	Asb.Cement/Concrete Fiber Glass	Low	140
99	38	133	545 (1)	1300	120	1976	Asbestos Cement	Reasonable	140
100	38	135	310 (1)	2000	100	1953/70	Cast Iron/Asb.Cement	Reasonable	110
101	39	40	688 (1)	1800	120	1976	Steel	High	100
102	39	102	328 (1)	1650	120	1979	Asbestos Cement	Reasonable	140
103	39	134	643 (1)	700	120	1976	Steel/Concrete	Reasonable	140
104	39	167	450	1650	120	1983	Asbestos Cement	Reasonable	140
105	40	137	551 (1)	700	100	1975	Asbestos Cement	Low	140
106	40	159	278 (1)	1600	120	1970/78	Asbestos Cement	Reasonable	140
107	41	42	450	2300	80	1975/79	Asbestos Cement	Very Low	140
108	41	57	320 (1)	2800	120	1958/81	Asb. Cement/PVC	Low	140
109	41	59	450	1250	80	1951	Cast Iron	Reasonable	80
110	42	60	500	1200	70	1951	Steel	Reasonable	80
111	43	44	900	1800	90	1969	Steel	Reasonable	80
112	43	60	750	2100	70	1970	Concrete	Very Low	140
113	44	45	507 (1)	1600	130	1976	Asbestos Cement	Reasonable	140
114	44	64	900	1200	90	1965	Concrete	Very Low	140
115	44	152	466 (1)	500	100	1938/48	Cast Iron	High	80
116	45	68	450	2800	130	1973	Asbestos Cement	Reasonable	140
117	45	154	500	1650	110	1951/60	Steel	High	100
118	46	47	277 (1)	1000	120	1963/67	Asbestos Cement	Reasonable	140
119	46	193	300	1700	130	1963	Asbestos Cement	Reasonable	140
120	47	68	450	2000	130	1977	Asbestos Cement	Reasonable	140
121	47	156	450	2200	120	1975	Asbestos Cement	Reasonable	140
122	48	49	750	2000	120	1973	Concrete/Fiber Glass	Reasonable	140
123	48	76	300	2000	120	1981/87	Asbestos Cement	Reasonable	140
124	48	183	450	1900	100	1977	Asbestos Cement	Low	140
125	48	184	300	2000	120	1976	Asbestos Cement	Reasonable	140
126	49	50	750	2050	120	1971	Concrete	Low	140
127	49	173	600	1300	130	1974	Concrete	Reasonable	140
128	49	184	411 (1)	2050	120	1972	Asbestos Cement	Reasonable	140
129	50	71	450	1800	120	1976	Asbestos Cement	Reasonable	140
130	51	52	750	1600	120	1974	Fiber Glass	Reasonable	140
131	51	72	390 (1)	1500	120	1959/74	Asb. Cement/PVC Fiber Glass	Reasonable	140

(table continued on next page)

Table 4-8. Continued

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
132	52	53	900	1050	100	1975	Concrete	Very Low	140
133	52	123	900	1000	100	1970	Steel	Reasonable	100
134	53	55	900	2200	110	1975	Concrete	Low	140
135	53	185	450	500	140	1976	Asbestos Cement	Reasonable	140
136	54	56	600	1100	50	1981/88	PVC/Concrete	Very Low	140
137	55	73	300	2900	140	1976	Asbestos Cement	Reasonable	140
138	55	122	387 (1)	1000	90	1961	Asbestos Cement	Low	140
139	55	177	1050	1800	110	1975	Steel	Reasonable	100
140	55	185	300	1600	140	1978	Asbestos Cement	Reasonable	140
141	56	70	399 (1)	2350	90	1978	Asbestos Cement	Low	140
142	56	75	1050	1950	90	1975	Steel	Reasonable	100
143	56	177	1050	2000	110	1975	Steel	Reasonable	100
144	57	58	472 (1)	1000	100	1976	Asbestos Cement	Low	140
145	57	137	510 (1)	1200	120	1975	Asbestos Cement	Reasonable	140
146	57	159	300	800	125	1970	Asbestos Cement	Reasonable	140
147	58	169	450	1150	100	1969	Asbestos Cement	Low	140
148	59	144	500	900	80	1955	Steel	Reasonable	80
149	59	169	496 (1)	1800	90	1956/66	Weld Stl/Asb.Cement	Reasonable	80
150	60	64	500	2500	75	1958	Steel	Reasonable	85
151	60	103	750	50	120	1972	Steel	High	80
152	60	144	500	1500	70	1955	Welded Steel	Reasonable	80
153	60	153	350	2200	80	1964	Asbestos Cement	Low	140
154	62	63	600	1800	90	1972	Fiber Glass	Low	140
155	62	142	600	1100	100	1972	Fiber Glass	Low	140
156	62	143	300	550	130	1976	Asbestos Cement	Reasonable	140
157	64	153	900	750	90	1964	Concrete	Very Low	140
158	64	154	500	1650	110	1958	Steel	High	80
159	65	66	600	800	110	1964	Steel	High	100
160	65	87	750	800	90	1967	Concrete	Very Low	140
161	65	153	900	900	90	1964	Concrete	Very Low	140
162	66	67	600	2000	120	1963	Steel	High	100
163	66	154	450	800	120	1961	Asbestos Cement	Reasonable	140
164	67	68	450	900	120	1974	Asbestos Cement	Reasonable	140
165	67	154	355 (1)	2100	130	1957/63	Asb. Cement/PVC Cast Iron	Reasonable	140
166	67	156	300	2050	130	1974	Asbestos Cement	Reasonable	140
167	67	157	600	1200	125	1974	Concrete	Low	140
168	69	94	750	50	50	1975	Concrete	Very Low	140
169	69	156	450	700	120	1975	Asbestos Cement	Reasonable	140
170	69	157	600	600	110	1974	Concrete	Low	140
171	69	158	642 (1)	1650	110	1975	Concrete	Low	140
172	71	175	317 (1)	4100	110	1983	Asbestos Cement	Low	140
173	72	74	450	1500	120	1980	Asbestos Cement	Reasonable	140
174	73	74	600	700	140	1978	Concrete	Reasonable	140

(table continued on next page)

Table 4-8. Continued

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
175	73	179	750	50	120	1978	Concrete	Low	140
176	74	185	600	800	140	1978	Concrete	Reasonable	140
177	75	130	1050	3500	110	1976	Steel/Concrete	Reasonable	120
178	76	79	300	1900	130	1977	Asbestos Cement	Reasonable	140
179	76	173	450	1350	130	1974	Asbestos Cement	Reasonable	140
180	77	80	450	700	110	1979	Asbestos Cement	Low	140
181	78	79	300	1700	130	1976	Asbestos Cement	Reasonable	140
182	78	173	300	1900	130	1974	Asbestos Cement	Reasonable	140
183	79	173	300	1700	130	1974	Asbestos Cement	Reasonable	140
184	80	81	300	2100	130	1974	PVC	Reasonable	140
185	80	125	300	1200	130	1980	Asbestos Cement	Reasonable	140
186	80	172	450	950	110	1979	Asbestos Cement	Low	140
187	81	124	600	1200	100	1984	Concrete	Very Low	140
188	81	172	629 (1)	800	120	1982	Concrete	Low	140
189	82	124	450	750	120	1984	Concrete	Low	140
190	83	104	1050	5150	120	1987	Concrete	Low	140
191	84	85	931 (1)	1000	80	1975	Concrete	Very Low	140
192	84	105	450	3200	105		Asbestos Cement	Low	140
193	84	141	300	1000	130	1973	Asbestos Cement	Reasonable	140
194	84	161	900	700	50	1975	Concrete	VeryLow	140
195	85	142	600	1100	110	1974	Fiber Glass	Low	140
196	85	163	900	1700	80	1975	Steel	Reasonable	100
197	85	165	300	1700	120	1978	Asbestos Cement	Reasonable	140
198	86	88	450	900	120	1971	Asbestos Cement	Reasonable	140
199	86	93	450	1700	110	1971	Asbestos Cement	Low	140
200	86	170	300	2000	100	1971	Asbestos Cement	Low	140
201	88	90	450	1350	110	1968	Asbestos Cement	Low	140
202	88	155	472 (1)	1500	120	1967	Asbestos Cement	Reasonable	140
203	89	155	450	1400	120	1967	Asbestos Cement	Reasonable	140
204	96	162	750	50	40	1976	Concrete	Very Low	140
205	97	100	1050	2000	110	1976	Steel/Concrete	Low	120
206	97	130	1050	1300	110	1976	Concrete	Low	140
207	97	162	750	150	70	1976	Concrete	Very Low	140
208	98	99	750	100	50	1969	Concrete	Very Low	140
209	98	191	750	50	50	1972	Fiber Glass	Very Low	140
210	99	192	750	50	50	1969	Concrete	Very Low	140
211	100	98	750	1400	100	1972	Fiber Glass/Steel	Reasonable	110
212	98	99	750	100	50	1969	Concrete	Very Low	140
213	98	191	750	50	50	1972	Fiber Glass	Very Low	140
214	99	192	750	50	50	1969	Concrete	Very Low	140
215	100	98	750	1400	100	1972	Fiber Glass/Steel	Reasonable	100
216	100	101	1050	2650	110	1978	Concrete	Low	140
217	101	132	1050	1250	110	1981	Concrete	Low	140
218	102	104	1050	7900	110	1981	Concrete	Low	140
219	102	132	1050	1150	110	1981	Concrete	Low	140

(table continued on next page)

Table 4-8. Continued

No.	Node	Node	Diameter (mm)	Length (m)	C Factor (2)	Year	Material	C Factor Classification	Suggeted C Factor (3)
220	102	134	600	1600	120	1980	Concrete	Low	140
221	105	166	310 (1)	2100	105		Asbestos Cement	Low	140
222	107	163	300	2000	120	1980	Asbestos Cement	Reasonable	140
223	107	164	450	600	100	1980	Asbestos Cement	Low	140
224	108	109	900	2000	80	1984	Concrete	Very Low	140
225	108	118	600	950	70	1983	Concrete	Very Low	140
226	109	186	500	1700	80	1973	Steel	Reasonable	140
227	110	176	300	1900	110	1962	Asbestos Cement	Low	140
228	112	171	300	1700	110	1962	Asbestos Cement	Low	140
229	113	171	474 (1)	2650	100	1962	Asbestos Cement	Low	140
230	114	115	400	900	80	1951	Welded Steel	Reasonable	140
231	114	180	300	1200	110	1951	Cast Iron	High	80
232	115	181	350	2400	110	1961	Asbestos Cement	Low	140
233	122	123	300	2500	110	1964	Asbestos Cement	Low	140
234	125	172	634 (1)	900	120	1982	Concrete	Low	140
235	130	131	300	2800	120	1976	Asbestos Cement	Reasonable	140
236	131	162	781 (1)	1500	120	1976	Concrete	Low	140
237	132	133	504 (1)	1600	120	1976	Asbestos Cement	Reasonable	140
238	133	134	400	1150	120	1968	Asbestos Cement	Reasonable	140
239	136	137	300	900	120	1962	Asbestos Cement	Reasonable	140
240	141	142	300	850	130	1972	Asbestos Cement	Reasonable	140
241	141	143	300	1750	130	1974	Asbestos Cement	Reasonable	140
242	141	161	225 (1)	750	130		Asbestos Cement	Reasonable	140
243	142	165	300	1100	130	1978	Asbestos Cement	Reasonable	140
244	144	143	313 (1)	1000	130	1962	Cast Iron/Asb. Cement	High	100
245	146	147	392 (1)	2600	110	1966/70	Asbestos Cement	Low	140
246	151	152	321 (1)	1000	110	1612/23	Cast Iron	Very High	80
247	157	158	300	2500	120	1977	Asbestos Cement	Reasonable	140
248	160	161	300	3750	130	1957/82	Asbestos Cement	Reasonable	140
249	160	166	250	3600	120		Asbestos Cement	Reasonable	140
250	163	164	900	1050	80	1980	Concrete	Very Low	140
251	163	165	300	1800	130	1978	Asbestos Cement	Reasonable	140
252	164	170	900	1200	80	1970	Concrete	Very Low	140
253	167	194	450	1650	120	1983	Asbestos Cement	Reasonable	140
254	168	169	363 (1)	950	110	1969	Asbestos Cement	Reasonable	140
255	168	194	600	2650	120	1984	Concrete	Low	140
256	174	186	750	4200	80	1973	Steel	Reasonable	80
257	180	181	300	1300	110	1951/60	Cast Iron	High	80

(based on the model used by the City of Edmonton Public Works)

(1) Equivalent diameters (section with different materials or diameters)

(2) Assumed C factor used in the model

(3) Based on manufacturers suggestion (ASCE, 1975; AWWA, 1979; 1980a; Lamont, 1981)

Figure 4-14. C-Factors of Edmonton's Water Distribution Network Model

(based on data from Table 4-8)

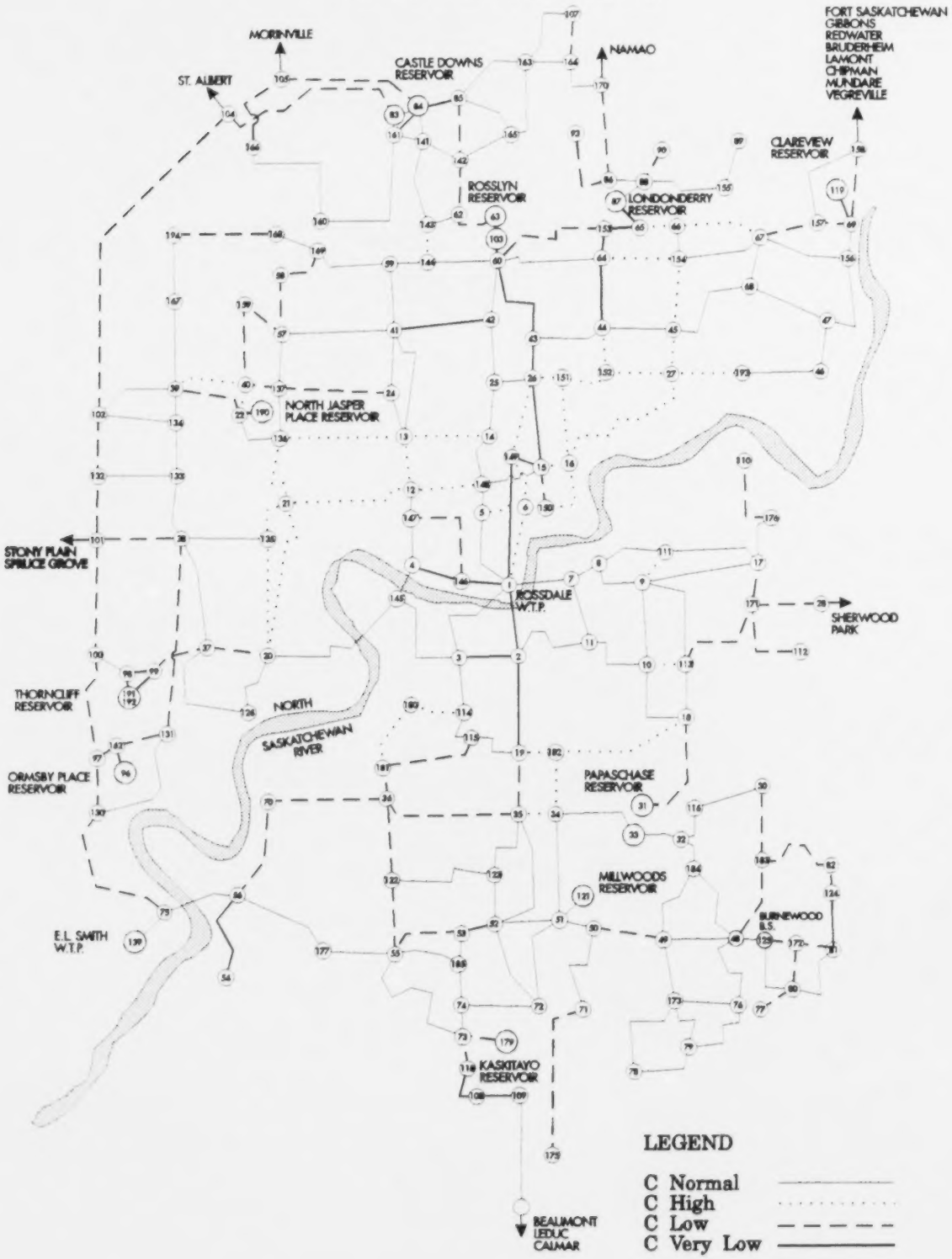
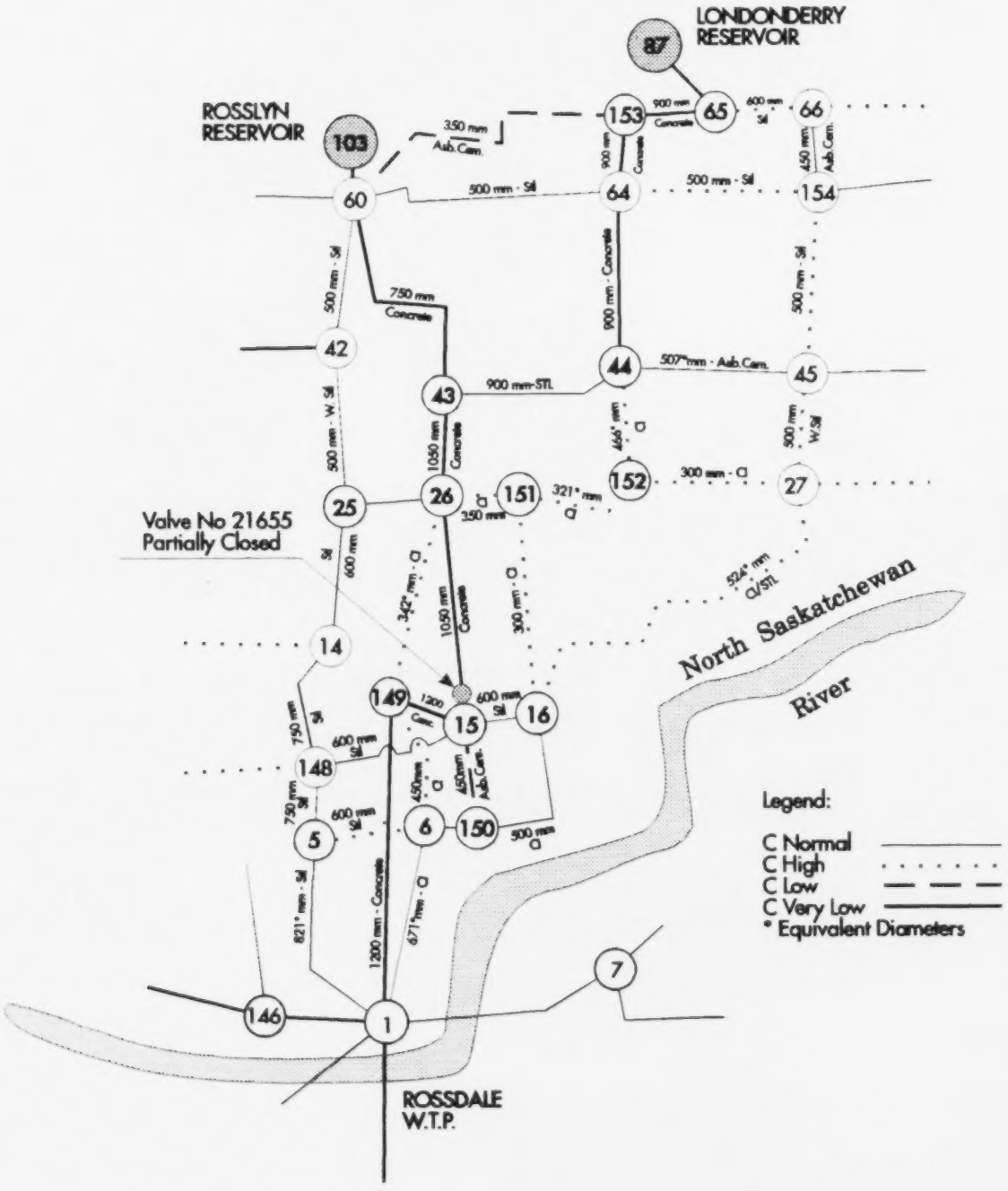


Table 4-9. Comparison Between C-Factors Used in the Model and Suggeted Values

Material	C-Factor Classification				
	Very-Low	Low	High	Reasonable	Total
Concrete Pressure Pipe	30	26		3	59
Asbestos Cement	15	14		67	96
Steel			12	33	45
Welded Steel			6	9	15
Lined Steel				2	2
Cast Iron			22	6	28
PVC		3		1	4
Fiber Glass (FRC)		5		3	8
Total	45	48	40	124	257

Figure 4-15. Area Affected by the Valve Partially Closed in Line II

(based on data from Table 4-8)



Analyzing Table 4-9, it is possible to verify the following:

- o C-factors for asbestos cement, steel, and lined steel pipes were reasonably in agreement with the suggested values.
- o C-factors for welded steel pipes were mixed between high and reasonable values.
- o C-factors for fiber glass reinforced pipes were mixed between low and reasonable values.
- o. C-factors for concrete pressure and PVC pipes were much lower than the suggested values.
- o C-factors for cast iron pipes were higher than the suggested values.

Based on information from the Water Branch, those C-factors were adjusted to calibrate the model. Walski (1985) described a common misunderstanding in calibrating a model as assuming water use as correct and adjusting C factors to match the pressure in the field. Based on this behavior, the model seems calibrated, however, it is not. On the contrary, new errors may have been introduced.

The analysis of Tables 4-8 and 4-9 and Figure 4-14 allowed to verify that some pipes in the model were carrying the water that other pipes should carry. This was verified in the analysis of Line II, which had a butterfly valve partially closed. Figure 4-15 shows a detail of the area affected by this valve. It is possible to verify that all concrete pressure pipes had C factors 80 or 70. Nevertheless, some unlined steel and cast iron pipes almost 70 years old had C-factor above 110. This shows that those metallic pipes had their C factor increased to carry the water that was not passing through the partially closed valve.

Waslki (1983a; 1983b; 1984; 1985; 1986) and Cesario and Davis (1984) described procedures for model calibration. One important point highlighted was the use of measured C-factors. The model has to mirror the field operation of the water distribution network. Therefore, only small adjustments should be applied to the pipe roughness, otherwise, the correction of one parameter may introduce errors in another one.

Summarizing, the model used by the Water Branch of the City of Edmonton is not representing the actual field situation. Unreal C-factors were used due to a misunderstanding in model calibration. Therefore, it is not possible to use the model for an accurate simulation of the network behavior.

Pipeline and pumping station design

Pipelines tested showed to be operating at very low flow rates. Furthermore, based on information from the Water Network Engineering, other pipelines proposed for head

loss testing (Table 3-6) could not be tested due to low operational flow rates. Considering the criterion suggested by Walski (1984) for pipeline design, a value normally adopted for pipeline selection is 1.8 m/s for peak demand. Water mains in the City of Edmonton are operating with velocities much lower than that even at peak hours. For example, Line XII had a maximum velocity of 0.43 m/s for a period of observation of 24 hours and an average of 0.35 m/s in the period.

The observation of the simulation model data, operational data, and collected information allowed to verify that almost all large diameter pipelines in the City of Edmonton were operating well below their capacity. If a pipeline which roughness was supposed to be high is transporting at low flow rate, the most economical solution for sure is not the construction of a new parallel line. Some issues may be verified as:

- detection by developing flow tests of partially or totally closed valves;
- detection by developing flow tests of debris jammed into the pipeline;
- deficiency in air removal from the pipeline;
- pumping capacity inadequate for the transportation of the required flow rate, and;
- pump efficiency not adequate for the system.

Another important factor detected during the development of the studies was a lack of design provision for points to develop flow and head loss tests in the network. Some basic procedures may be adopted for providing adequate testing points in future design, as follows:

1. select chambers for flow and pressure measurement in all new pipelines with diameter superior to 600 mm;
2. tap and install gate valves in the tap in the chambers as follows:
   - include in the testing chamber at least one 50 mm gate valve to measure flow and pressure;
   - position the tap in the chamber conveniently to allow measuring equipment manipulation and operation, and;
   - position the chamber not to be affected by fittings in the pipeline

Pipeline maintenance and operation

The development of field tests in the water distribution mains showed to be a good tool not only to obtain actual information for design and modelling, but also to detect problems in the network. Head loss tests and flow tests may be developed to help the

detection of those problems, contributing for a better system operation and saving money.

For illustration, it is possible to report the valve found partially closed in Line II. This valve was in that position for more than two years, requiring additional pumping to feed a large area of North-East Edmonton. During this period a large amount of extra money was spent to supply the pumps with energy that was not required.

Pipeline testing

Field tests showed their importance in various sectors of a water distribution system. However, the validity of the results depended on the accuracy of the obtained parameters. The field tests here developed allowed to verify that the accuracy of the results depended on the following issues:

1. errors in flow rate:
    - o accuracy of the flowmeter;
    - o calibration of the flowmeter;
    - o position of the flowmeter relative to fittings and valves, and;
    - o correct determination of the position of the average velocity in the pipe cross section.
2. errors in head loss:
    - o accuracy of the pressure measurement device;
    - o calibration of the pressure measurement device, and;
    - o position of the pressure measurement device relative to fittings and valves.
3. error in pipe length:
    - o errors introduced in field measurement (in the "as builts" drawings);
4. errors in pipe diameter:
    - o impossibility of obtaining actual pipe diameter, and;
    - o nominal diameter very different from actual diameter.

Flow rate and pressure measurement errors were minimized by doing a correct calibration and set up of the equipment. Therefore, the errors depended almost exclusively on the accuracy of the equipment and its position relative to fittings and

valves. Existing noise in the results was also reduced by applying to them statistics tools for data analysis. Errors in head loss were also minimized by using results of tests in pipelines with difference between pressure heads higher than 1 m.

Errors in length measurement were very improbable. Lengths from "as builts" drawings were double checked in the field and in the office, therefore, small risk was considered for errors.

Correct pipe diameters were an unknown. Issues such as different pipe diameter and pipe material inside valve chambers required the use of the nominal diameter instead of the actual tested pipeline diameter. The accuracy of the obtained C-factors depended on how close the nominal pipe diameters were to the actual pipe diameters. Therefore, these C-factors are correct for the tested pipes, and they showed a tendency of behavior of the pipe roughness in the types of materials tested. However, this roughness tendency has to be confirmed by developing further tests in large diameter water mains.

The effect of each parameter in the calculation of C-factors is shown on Table 4-10. Each of the four parameters was separately analyzed, increasing and decreasing its value by 5 and 10 percent. Then, the C-factor was recalculated, keeping three parameters with the field test results and substituting each analyzed parameter at a time. Table 4-10 allows to verify the much higher effect of the diameter variation in the calculation of the C-factor. As expected, the analysis of equation 2-21 showed the parameter diameter risen to an exponent 2.63 while others had smaller exponents. This high effect of the diameter in the calculation of C-factors showed that an extra effort may have to be applied in the determination of the correct pipe diameter in future tests.

Practical conclusions

1. C-factors used in the model of the Water Branch of the City of Edmonton were not matching the field situation for some types of pipes;
2. C-factors used for concrete pressure pipes were very low;
3. C-factors used for some metallic pipes were very high;
4. model did not reflect the field situation due to wrong calibration;
5. pipelines tested and other pipelines in the City of Edmonton network were operating far from their ideal and economical capacity;
6. if pipelines cannot transport design flow, field tests are helpful tools for the determination of the problem;
7. it is necessary to provide adequate testing chambers in the design phase of the pipeline.

8. variation in the assumed diameter value affects substantially the calculated C-factor;

9. whenever possible measure the actual pipe diameter in the field;

10. developed tests showed a tendency of the behavior of roughness in steel and pressure concrete pipes, and;

11. new tests in other large diameter pipes have to be developed to confirm these tendencies.

Table 4-10. Sensivity Analysis of the Parameters Used in the Calculation of C-Factors

Variation of the Parameter (%)	Parameter Analyzed							
	Diameter (D)		Length (L)		Head Loss (h)		Flow Rate (Q)	
	Calculated C (2)	Variation of C (4) (%)	Calculated C (2)	Variation of C (3) (%)	Calculated C (2)	Variation of C (3) (%)	Calculated C (2)	Variation of C (3) (%)
Line I								
0 (1)	107 (3)		107 (3)		107 (3)		107 (3)	
- 5	123	14	104	-3	110	3	102	-5
+5	94	-12	110	3	104	-3	113	5
- 10	142	32	101	-6	114	6	97	-10
+10	83	-22	113	5	102	-5	118	10
Line IX								
0 (1)	166 (3)		166 (3)		166 (3)		166 (3)	
- 5	189	14	160	-3	169	2	157	-6
+5	145	-13	169	2	161	-3	173	4
- 10	217	31	156	-6	174	5	148	-11
+10	128	-23	174	5	157	-6	181	9

(1) Values obtained in the field tests

(2) C calculated keeping all parameters obtained in the field tests, but changing the parameter analized

(3) C = Co (C-Factor calculated with the parameters obtained in the field tests)

(4) 100 * [(C/Co) - 1]

5. CONCLUSIONS

The most important conclusions reached in the studies for measuring C factor in large diameter pipelines were:

1. the tested concrete pressure pipes were operating as smooth pipes with C-factors always above 150;

2. C-factors in the tested concrete pressure pipes were above the suggested values in the literature review.;

3. C-factor in the tested steel pipe matched the suggested values in the literature review;

4. treated water attack in the tested steel pipe (16 years old) was estimated as moderate to appreciable;

5. the hydraulic grade line method for measurement of head loss in large diameter mains is more accurate for head losses above 1 m;

6. lines in which the measured head loss is lower than 1.0 m the parallel pipe method has to be used to reduce pressure measurement error;

7. the position of the valve chamber in which pressure and flow have to be measured is very important when positioning taps to measure these parameters;

8. pressure and flows measured in valve chambers close to appurtenances had greater variation of the measured parameters due to higher turbulence;

9. flow measurement in control valve chambers required the definition of the point in which the average velocity was found to measure flow;

10. the water distribution network simulation model used by the Water Branch of the City of Edmonton did not represent the field situation;

11. C-factors for concrete pressure and cast iron pipes used in the model did not match the field situation;

12. installation of control valves with diameter smaller than the pipeline introduced turbulence and increased head losses in the pipeline;

13. many pipelines in Edmonton's water distribution network are operating with flow rates well below the ideal and economical flow rate;

14. tests developed in pipelines help the location of problematic points in pipes presenting unexpected low carrying capacity (determination of closed valves, debris jammed into the pipe, and leakages);

15. the variation of the diameter used in the calculation of the C-factors affects the calculated C-factor much more than the variation of other parameters;

16. when the actual pipe diameter is unknown, the calculated C-factor is correct for the tested pipe with the used diameter, but not for any pipe of same material and age;

17. uncertainty in diameter measurement shows a tendency of the C-factor for a determined type of pipe material and age, and;

18. new tests need to be developed to confirm the tendency of C-factors determined using diameters not known.

6. RECOMMENDATIONS

The study here developed allowed to draw conclusions about the C-factors being used by the Water Branch of the City of Edmonton and the procedures to obtain the C-factors. Some recommendations are provided below to improve accuracy of the C-factors used in design and modelling.

1. A constant testing routine must be followed to allow a good record of pipe C-factors.

2. Records of the average ratio nominal diameter to actual diameter should be kept for new pipelines being constructed.

3. An extra effort should be developed to obtain the average ratio nominal diameter to actual diameter in existing pipelines to be tested in future head loss tests.

4. New tests in steel pipes should be developed using the hydraulic grade line or parallel pipe method to confirm the tendency of the C-factors obtained in this study:

   - o Line IV;
   - o Line V;
   - o Line XI;
   - o Line XIV;
   - o Line XV, and;
   - o other lines that meet the set criteria.

5. New tests in concrete pressure pipes should be developed using the parallel pipe method to confirm the tendency of the C-factors obtained in this study. If flow in the suggested lines may be increased to allow a head loss above 1 m, the grade line method may be used.

   - o Line II;
   - o Line III;
   - o Line VII;
   - o Line VIII;
   - o Line X;
   - o Line XIII;
   - o Line XVI, and;
   - o other lines that meet the set criteria.

6. New tests in concrete pressure pipes that may use the hydraulic grade line method should be developed to confirm the tendency of the C-factors obtained in this study:
   - o Line II;
   - o Line III;
   - o Line XIII;
   - o Line XVI, and;
   - o other lines that meet the set criteria.

7. Taps used for flow and pressure measurements in testing existent lines should be positioned to minimize the effect of fittings and valves.

8. Newly designed pipelines must provide chambers adequate for measuring flow and pressure in head loss and flow tests.

9. C-factors in the water distribution simulation model of the City of Edmonton Public Works should be reviewed and adjusted to match field test values.

10. The water distribution model should be calibrated before meaningful simulation can occur.

11. C-factors should not be changed in the model to match field pressures with model pressures.

12. Pipelines found to have a low capacity in relation to the theoretical value should be tested to locate the problem instead of assuming the pipeline has a low carrying capacity.

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